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AMERICAN SOCIETY

OF

CIVIL ENGINEERS.

(INSTITUTED 1852.)

VOL XLIII.

JUNE, 1900.

NEW YORK:
PUBLISHED BY THE SOCIETY.

1900.

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Page 233, Fig. 16: For 1894–5, read 1884–5.

Vol. XLIII. JUNE, 1900.

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No. 865.

IMPACT TESTS OF STRUCTURAL STEEL.

BY S. BENT RUSSELL, M. Am. Soc. C. E.

PRESENTED OCTOBER 4TH, 1899.

WITH DISCUSSION.

In this paper will be given the results of some impact tests of small specimens of wrought iron and soft steel. The tests were made by a new method, under which the specimens were broken by tensile stress.

The matter to be presented will be given in its natural order, beginning with a few words on the need of tests of this character. The theory of the methods will then be given, followed by a description of the appliances used in the tests. The results of the experiments will then be stated, and, lastly, the conclusions that may be drawn from the observations.

Now, as to the need of study on the effect of impact on steel, if we leave out the engineers, the average man would be found to think that the strength of steel was its most important quality. Engineers, however, do not take the same view of the matter. To give an illustration, a well-known and experienced engineer once remarked to the author, that in ordinary engineering work he cared but little about the tensile strength of the steel he was to use, as compared with the general

reliability of the metal, *i. e.*, its uniform toughness and ability to withstand shocks and distortion.

This view is, without doubt, held by the majority of experienced engineers. In fact, it may be said, that among engineers the importance of shock resistance is generally admitted. Most engineers, however, will be found content with the methods now in use for determining the shock resistance of steel. They will contend that any skilful blacksmith can test a sample of steel and say whether or not it is tough and reliable.

There is, on the other hand, among engineers and scientific men, a rapidly growing feeling as to the importance of impact tests of structural materials, made, not with the blacksmith's methods and muscular sense, but in accordance with the stricter rules of experimental science. No argument is needed, then, to show that at this time studies of the effect of impact on steel are quite in order, and that the first step is to seek a satisfactory method of testing steel. The hope of finding a method which would be in advance of present practice has led to the experiments herein given.

Theory of the Methods Used.

The following description, with Figs. 1 to 5, shows how the new design may be evolved, step by step, from the old type of impact machine.

Fig. 1 shows in outline the familiar form of impact testing machine, where the test bar rests on two rigid supports and is struck in the middle by a falling weight. Owing to the rigidity of the knife-edges, *K*, the energy of the blow must be absorbed in deflecting the test piece *F*.

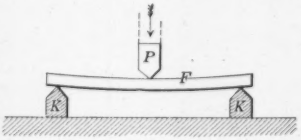


Fig. 1.

Suppose, now, a set of conditions where the bar *F* is comparatively rigid, and, in place of one of the knife-edges, there is a yielding support or spring *S*, as shown in Fig. 2. The energy of the blow from the falling weight *P* will now be absorbed in compressing the spring *S*. This will be strictly true only where the bar *F* and the remaining knife-edge *K* are perfectly rigid.

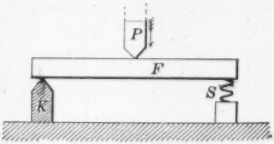


Fig. 2.

Fig. 3 shows the same arrangement, except that the spring S acts in tension and is supported by a bracket B, and, if the bar F, the knife-edge K and the bracket B are perfectly rigid, the entire energy of the blow from P will be absorbed in stretching the spring S.

In order to lessen the strain on the bar F and on the knife-edge K, the point of application of the blow may be changed, as shown in Fig. 4.

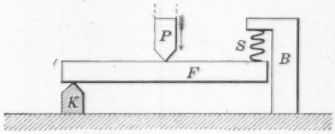


FIG. 3.

Now, if we substitute in place of the spring S a test bar of metal we have an impact testing machine which will show the tensile resistance to impact of the material.

Let us now make another change in the arrangement. Place the knife-edge K vertical and substitute for the falling weight a pendulum swinging on a horizontal axis above and parallel to the fork-bar F, so that the pendulum P will strike a horizontal blow on the bar F, as shown in Fig. 5, which is a ground plan. Let the knife-edge K and the bracket B rest against independent piers, so that, when the specimen breaks, the fork-bar F will swing to one side and allow the pendulum to pass.

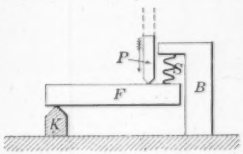


FIG. 4.

With this arrangement the pendulum may be raised to a given height and released. It will strike the bar F, tear the specimen in two and rise after the blow to a certain height. The difference between the height through which the pendulum fell and the height to which it rose after the blow shows the amount of energy absorbed in tearing the specimen. Fig. 5 may be said to show diagrammatically the form of machine used in the experiments herein described. From the foregoing it will be understood that by this method the specimens are broken by direct tension, and hence the tests may be called tensile impact tests.

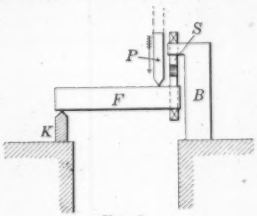


FIG. 5.

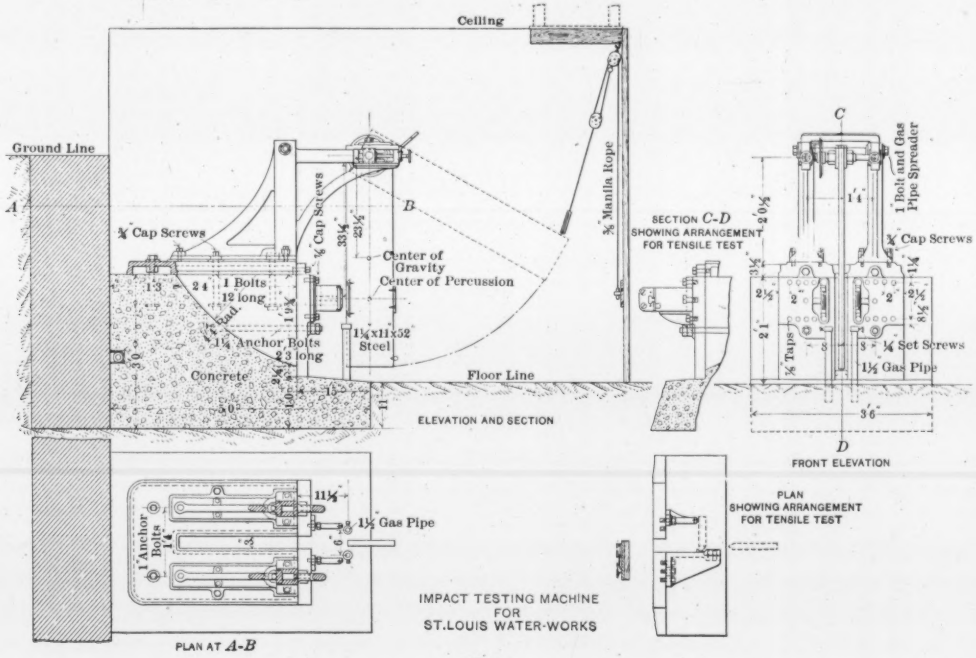

Ceiling
Ground Line
A
¾ Cap Screws
¾ Cap Screws
¾ Manila Rope
B
Center of Gravity
Center of Percussion
1 Bolts 12 long
Rod
1¼ Anchor Bolts 23 long
Concrete
1¼x11x52 Steel
Floor Line
ELEVATION AND SECTION
SECTION C-D SHOWING ARRANGEMENT FOR TENSILE TEST
C
1 Bolt and Gas Pipe Spreader
¾ Cap Screws
⅜ Taps
⅜ ¼ Set Screws
1½ Gas Pipe
D
FRONT ELEVATION
PLAN SHOWING ARRANGEMENT FOR TENSILE TEST
Anchor Bolts
1½ Gas Pipe
PLAN AT A-B
IMPACT TESTING MACHINE FOR ST. LOUIS WATER-WORKS

FIG. 6.

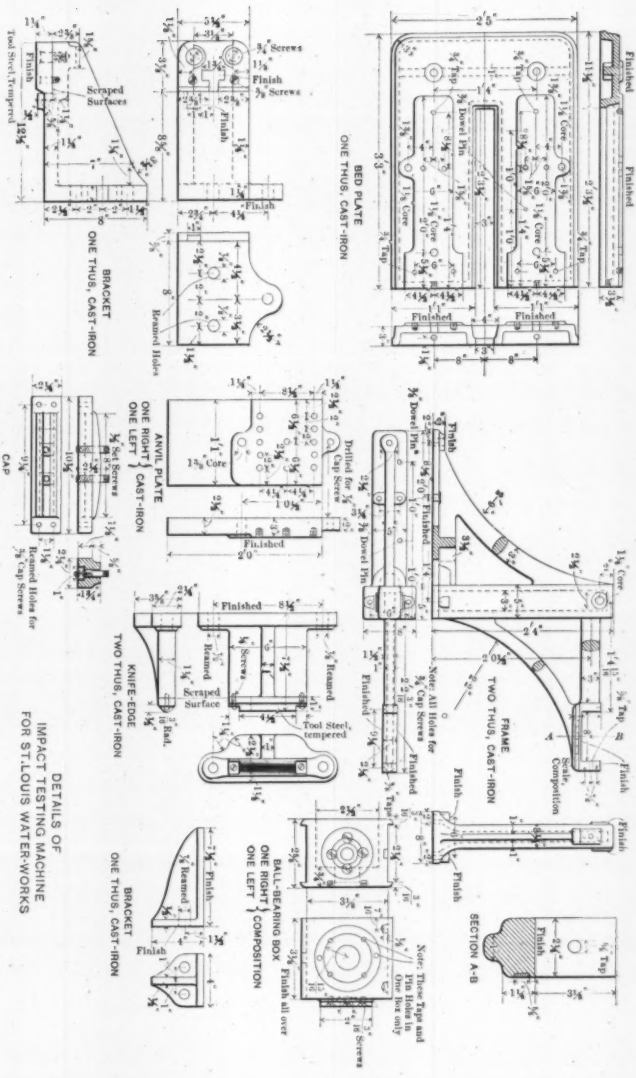


Fig. 7.

DETAILS OF
IMPACT TESTING MACHINE
FOR ST. LOUIS WATER-WORKS

The Testing Machine.

The testing machine used in the experiments has been partly described and illustrated in the *Transactions* of this Society.* The original machine was devised by the writer for making transverse tests by impact. The attachments by which tensile impact tests could be made were also devised by the writer. Mr. William F. Schaefer rendered valuable assistance in the execution of the scheme. The attachments for tensile tests were made in March, 1898. Figs. 6 and 7 show the form and some of the details of a new testing machine now being built for the St. Louis Water-Works Extension. In principle, the two machines are alike.

The new machine will have a pendulum or hammer of forged steel, rectangular in form and weighing about 200 lbs. It will hang on a

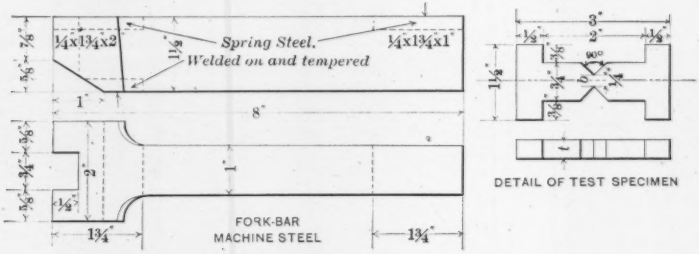


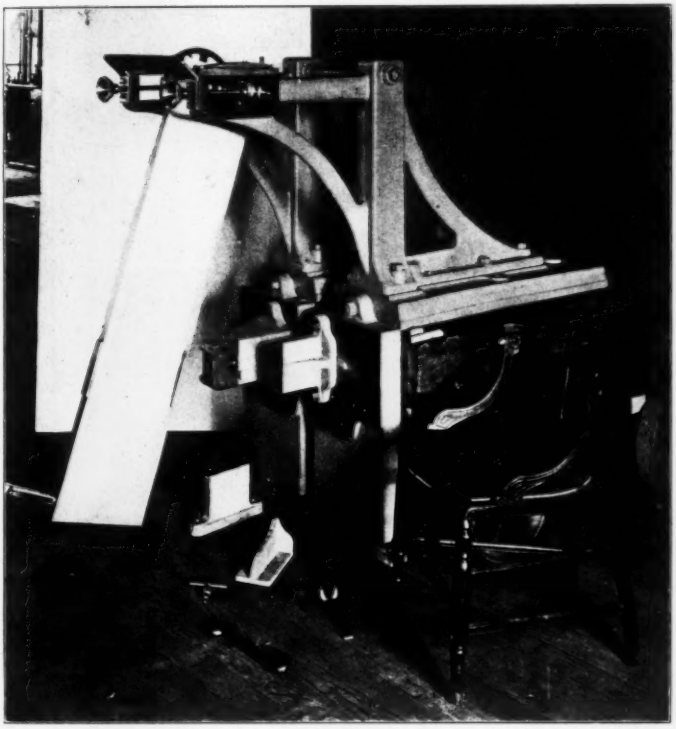
FIG. 8.

horizontal shaft resting in ball bearings. The striking edge of the pendulum and the fixed knife-edge will be of tempered tool steel.

The machine will be set up in a basement room, and the concrete anvil-block will abut against a stone cellar wall, below the level of the earth outside. By this arrangement the pendulum will swing clear of the floor of the room, and everything will be quite accessible and convenient. Plate I shows the frame and mechanism as assembled in the shop.

Fig. 8 gives the dimensions of the lighter fork-bar used. One of the fork-bars was made of machinery steel with faces of tool steel welded on at all bearings. These faces were tempered and then ground true. The other fork-bar was made of tool steel throughout, and tempered and ground at the bearings.

* "Experiments with a New Machine for Testing Materials by Impact." By S. Bent Russell, M. Am. Soc. C. E. *Transactions*, Am. Soc. C. E., Vol. xxxix, p. 237.



IMPACT TESTING MACHINE.

The bracket used is of cast iron. It is very heavy and rigid and is provided with a slotted plate of tempered steel. This plate receives the T-head of the specimen, and all bearing surfaces are ground true. There is a small shelf on the face side of the bracket which supports the fork end of the fork-bar when in position for a test. The other end of the fork-bar is held up by an adjustable support. When in position the axis of the fork-bar is on a level with the center of percussion of the pendulum.

The 7½ by 4-in. bracket, shown in Fig. 7, is used to support the heavy bracket while it is being bolted to the anvil plate.

In Fig. 6 two arrangements of the machine are shown. The first is for transverse impact tests, where two knife-edges are used.* The second is for tensile impact tests, where the bracket and fork-bar are used with one knife-edge only. In Plate I the parts are shown as set up for tensile tests.

In building these machines great care is taken to insure rigid supports. A solid concrete foundation is used, a heavy anvil plate is carefully bedded against the concrete, the seats of the bracket and knife-edge are planed off true, and the tempered steel striking plates are ground true on the seat and rest against scraped surfaces.

This care is necessary in order that there may be no springing in the joints when the blow is struck. The rule is that all fixed parts must be heavy and rigid, and all joints perfectly fitted and firmly bolted.

The hammer used in the experiments weighs 103 lbs., or about half what the new hammer will weigh. The relative proportions of its dimensions are the same as in the new hammer.

It is important that the hammer should strike on its center of percussion, so that there may be no blow on the trunnions. Where the hammer is of simple form, the center of percussion is readily computed.† After the hammer is set up, the center of percussion may be verified by timing the oscillations, the period of which should be the same as that of a simple pendulum, whose length is equal to the distance from the axis to the center of percussion.‡

The center of gravity of the pendulum or hammer is found by trial before mounting, and, at the same time, the hammer should be

* *Transactions*, Am. Soc. C. E., Vol. xxxix, p. 237.
† Johnson's "Materials of Construction," Art. 293.
‡ Rankine's "Applied Mechanics," Articles 544 and 607.

weighed, as the force of the blow is to be measured by the fall of the center of gravity in inches and the weight of the hammer in pounds. The pendulum is, of course, provided with an attachment for reading the height through which the center of gravity falls. The height through which the center of gravity rises after passing its lowest point is also shown.

We are now provided with a machine for breaking specimens in tension with a single blow. Before making actual tests, however, it is in order to consider what results will be obtained. In this paper the resilience of a specimen will be understood to mean its shock resistance or stopping power; that is, the amount of energy or work that will be required to rupture it. This energy will be expressed in inch-pounds.

If we raise the hammer 2 ins. and let it fall so as to break a specimen, and the hammer rises to a height of 1 in. after breaking the specimen, we would say that, as the hammer weighs 103 lbs., 103 inch-pounds of energy have been absorbed, so that the apparent resilience of the specimen is 103 inch-pounds. The loss due to friction in swinging is easily allowed for, therefore it may be considered that there is no friction.

In addition to breaking the specimen, there has been energy absorbed in other ways. The most important of these are the inertia and the springing of the fork-bar.

The Determination of Errors.

Problem: To Find the Loss due to the Inertia of the Fork-Bar.—According to Hodgkinson, the observed resilience is to the true resilience as $\frac{I + W}{W}$, when W is the weight of the hammer and I is the inertia of the test bar.* The problem is, then, to find I.

To simplify the problem we will assume that the fork-bar or the bar to be struck is a rectangular prism having its axis or fixed point at one end x, Fig. 9, and that it is struck at the other end. The length of the bar we will call r', and we will consider that the thickness and width of the bar are small enough to be neglected, or, in other words, that the bar is a straight line revolving about one end and having weight, but no width or thickness.

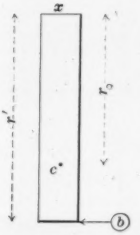


Fig. 9.

* *Transactions*, Am. Soc. C. E., Vol. xxxix, p. 239.

Let W_1 = the weight of the bar.

Let us now substitute, for our rectangular hammer striking on its center of percussion, an equivalent simple pendulum. Let b equal the weight of this simple pendulum.

We now have the case of the familiar ballistic pendulum; treating the bar, whose length is r', as the ballistic pendulum, and the weight b as the projectile (see Fig. 9).

The formula for this case will be found in Rankine's "Applied Mechanics."* Taking the same notation as Rankine:

Let v = the velocity of the weight b before striking;

r_o = the distance from the axis x to c, the center of gravity of the weight and bar together;

W = the weight of the ball b and the bar together;

l = the length of a simple pendulum equivalent to the combined mass revolving about the axis x;

$B = \frac{W r_o}{l}$ = the portion of the joint weight to be treated as if concentrated at the center of oscillation, or a distance l from x;

V = velocity of the center of oscillation of the joint mass at the instant of perfect contact, when both bodies are moving at the same velocity;

g = the acceleration due to gravity.

Professor Rankine shows that $\frac{B\,V^2}{2\,g} = \frac{b\,v^2}{2\,g} \cdot \frac{b\,r'^2}{B\,l^2}$.

To find the value of $\frac{b\,r'^2}{B\,l^2}$:

Let W_1 = the weight of the bar alone;

Let I' = the moment of inertia of the joint mass = $(\frac{1}{3}\ W_1 + b)\ r'^2$;

Let $\rho^2 = \frac{I'}{W}$ = the square of the radius of gyration of the joint mass; $l = \frac{\rho^2}{r_o}$.†

Substituting and cancelling out we find

$$\frac{b\,r'^2}{B\,l^2} = \frac{b}{\frac{1}{3}\ W_1 + b} \qquad (1)$$

which is the desired solution for the case of a hammer in the form of a simple pendulum.

*Article 607, p. 549. † Rankine, Art. 581.

To get this result in terms of known properties of the rectangular hammer: $b = \frac{W_2 R_2}{l_2}$.*

Where W_2 = weight of hammer;

R_2 = distance from axis of hammer to center of gravity of same; and

l_2 = distance from axis of hammer to center of percussion of same.

Substituting in (1) we get:

$$\frac{b r'^2}{B l^2} = \frac{1}{1 + \frac{W_1 l_2}{3 W_2 R_2}}; \text{ hence } \frac{B V^2}{2 g} = W_2 h \frac{1}{1 + \frac{W_1 l_2}{3 W_2 R_2}} \quad \ldots . (2)$$

= effective energy,

where h = the fall of the center of gravity of the hammer.

Let $W_2 h = R'$, and $\frac{B V^2}{2 g} = R$.

Then, from equation (2),

$$\frac{R'}{R} = \frac{1 + \frac{W_1 l_2}{3 W_2 R_2}}{1} = \frac{\frac{W_1 l_2}{3 R_2} + W_2}{W_2} = \frac{I + W}{W},$$

where $I = \frac{W_1 l_2}{3 R_2}$ and W_2 is written as W, thus giving the value of I in Hodgkinson's rule.

Conclusion.—The inertia of the fork-bar may be taken as one-third of the weight of the bar by the ratio $\frac{l_2}{R_2}$, or the ratio of the radius of the center of percussion to the radius of the center of gravity of the hammer.†

This rule is, of course, only approximately true for a bar of the shape actually used. The true correction for the bar could be found by the same line of reasoning, taking all the dimensions into account.

In the method above given no account is taken of the deflection of the fork-bar, which is assumed to be inconsiderable.

The rule may be applied to one of the fork-bars used in the experiments as follows:

Weight of fork-bar No. 1 = 6.39 lbs.;

Weight of hammer = 103 lbs.;

R_2 = 18 ins.;

l_2 = 25.75 ins.

* Rankine, Art. 607.

† Compare Merriman's "Mechanics of Materials," Articles 103 and 111.

As $W_1 = 6.39$, $\frac{W_1 l_2}{3 R_2} = \frac{6.39 \times 25.75}{3 \times 18} = 3.047 = I$, or the inertia of the bar in Hodgkinson's rule; and as $W = 103$,

$$\frac{I + W}{W} = \frac{3.047 + 103}{103} = \frac{102.96}{100}; \text{ hence } \frac{W_2 h}{R} = \frac{102.96}{100}; \text{ or}$$

the total energy absorbed is nearly 3% greater than the resilience of the specimen, the difference being due to the inertia of the fork-bar.

As the error is directly proportional to the weight of the fork-bar, we would have with a fork-bar of half the weight an error of less than 1½ per cent.

With any hammer of similar proportions, where the weight of the fork-bar is less that one-tenth of the weight of the hammer, the error due to the inertia of the fork-bar will be less than 5 per cent.

Problem: To Find the Error due to the Spring of the Fork-Bar.—We will consider the fork-bar as a beam supported at the ends and carrying a concentrated load. We first find the load P, which is determined by the maximum pull on the specimen at S_1 and the relative distances $S_2 P$ and $P S_1$, Fig. 10. From the dimensions of the beam and the modulus of elasticity of the metal we may find the deflection y of the beam at the loaded point. With all dimensions given, we find that by the accepted formulas the load P is directly proportional to the reaction S_1 and the deflection y is directly proportional to P. Hence y is directly proportional to S_1, or $y = K' S_1$, where K' is constant.

FIG. 10.

The energy absorbed in producing this deflection is

$$R' = \frac{P y}{2} = \frac{K'' S_1 y}{2},$$

where K'' is constant. Substituting for y we get $R' = K S_1^2$, where K is constant, which means that the loss of energy in springing the fork-bar is proportional to the square of the maximum pull on the test specimen. K may be readily computed for given conditions by well-known formulas.

In this way the value of K for the light bar used in these experiments has been computed to be equal to 0.0000001106.

With this value, taking a specimen having an ultimate strength of 10 000 lbs. for example, we find that the springing of the bar will absorb the energy: $R' = K S_1^2 = 11.06$ inch-pounds, where $S_1 = 10\,000$

lbs. If the supposed specimen absorbed 400 inch-pounds in breaking, we would have an error of $\frac{11.06}{400} = 2.76$ per cent.

This may be considered an average case. If the specimen should have half the strength and half the resilience above given we would have half the percentage of error, or 1.38 per cent.

As a stiffer bar may be used with stronger specimens, it would seem that with metal of ordinary strength and ductility there should be no difficulty in keeping the proportionate error below 5%, and that without using so heavy a fork-bar as to involve a large error from inertia.

TABLE No. 1.—RESILIENCE OF WROUGHT IRON—EFFECT OF DECREASING THE WEIGHT OF THE FORK-BAR.

Kind of wrought iron.	RESILIENCE IN INCH-POUNDS PER SQUARE INCH OF SECTION AT NICK. R_2		Gain, per cent.
	With fork-bar weighing 6.39 lbs.	With fork-bar weighing 3.85 lbs.	
Tennessee common.....	2 115	2 165	2.4
Tennessee charcoal.....	2 885	3 046	5.5
Average gain.....	.....	.....	3.95

NOTE.—All the specimens of each metal were of the same form and dimensions, and were cut from the same bar. Each value of R_2 is the average of six tests.

Table No. 1 gives the results of experiments made to learn the effect of decreasing the weight of the fork-bar when other conditions are kept as nearly uniform as may be. The results show that the energy absorbed in breaking a specimen is greater with the light bar. This would seem to indicate that the error due to the springing of the fork-bar is greater than the error due to the inertia effect. The difference, however, may be due to variations in the metal.

Without further investigation, it seems fair to presume that the inherent errors of this method of testing need not exceed 10%, or, in other words, over 90% of the blow should be absorbed by the specimen itself.*

In making comparative tests with the same fork-bar, the error due to inertia is of no effect. In the results of the experiments no correction has been made for either the inertia or spring of the fork-bar. It is thought that, were all the corrections made, the comparative values would not be changed materially.

* See *Transactions*, Am. Soc. C. E., Vol. xxxix, p. 244.

Preparing Specimens.

Taking up first the design of the specimen bars, we find that on account of the great toughness or resilience of steel, it is necessary to test but a small volume of the metal. As it is difficult to confine the effect of a blow to a definite volume of metal, a nicked section was adopted, a standard nick of simple form being used.

In Fig. 8 is shown the dimensions of the specimen bars used. As they are all cut from flat sections, all the dimensions are constant, except the thickness t.

To allow for the varying thickness, the resilience of a specimen is divided by the area of the cross-section at the nick, and the result is called the resilience per square inch. The results are believed to be comparable.

It should be remembered, of course, that a test specimen of different form would not give the same resilience per square inch. Objection may be made to the use of any particular form of nick, but it is believed that tests made with the form adopted give a good idea of the quality of the metal. The blacksmith judges a bar of steel by nicking it and breaking it with his hammer and anvil. We are simply improving on his method by using a nick made to gauge, and measuring the energy of the blow. It may be added that it is not unlikely that the sharpness of the nick may result in a greater range of resilience values than would be obtained could the impact tests be made on a prism 6 or 8 ins. long, such as is tested in ordinary determinations of tensile strength and ductility. If this is true, then the form of nick should be made to suit the conditions which the metal tested will meet in service.

It is in order, perhaps, to note here that in the *Transactions** of the Society will be found the results of previous experiments made by the writer, in which nicked specimen bars of steel were broken by a single blow. In these previous experiments the bars were broken transversely. The great range in the values of resilience of steels tested in this manner led the writer to think that transverse breaking gave an unfair advantage to the softer steels. Hence, this study of impact tensile tests with nicked specimens of steel.

The specimen bars used in the experiments were cut to shape in a milling machine, as experience showed this to be the best way to secure sharp and uniform nicks. The milling cutters were ground in a

* *Transactions*, Am. Soc. C. E., Vol. xxxix, p. 237.

universal cutter and tool grinder, so as to secure a perfect cutting tool. A number of specimens are usually cut together on the milling machine.

Breaking Specimens.

The specimens are numbered and then calipered with a micrometer. A specimen bar is then placed in the bracket slot, and the fork-bar adjusted. The hammer is raised to a given height and released. It falls, strikes the fork-bar and breaks the specimen. The height to which the hammer swings is recorded. The proper correction for friction of the hammer is noted, and the observer is ready for another specimen. Table No. 2 shows the form of record kept.

TABLE No. 2.—Impact Test.—Office of Water-Works Extension. —St. Louis, October 27th, 1898.—Specimen of Wrought Iron (Tenn. Char.) Taken from Iron Company.—Tested for Tensile Resilience, with Results herewith Appended.

Lot No. 4. Weight of fork-bar, 3.85 lbs.

Experiment No.	Melt No.	Bar No.	Initial fall of hammer, in inches. F.	Rise after blow, in inches. S.	Correction for friction, in inches. C.	Thickness of plate, in inches. t.	Width at nick, in inches. b.	Area, in square inches at nick. $A = tb$.	Effective fall of hammer. $H = F - (S + C)$.	Total resilience, in inch-pounds. $R = 108\ H$.	Resilience per square inch, in inch-pounds. $R_2 = \frac{R}{A}$
1 112	*B*	1	1.80	0.13	0.03	0.231	0.230	0.05313	1.64	168.92	3 179
1 113	*B*	2	1.80	0.21	0.03	0.231	0.230	0.05313	1.56	160.68	3 005
1 114	*B*	3	1.80	0.18	0.03	0.233	0.230	0.05359	1.59	164.77	3 074
1 115	*B*	4	1.80	0.17	0.03	0.232	0.231	0.05359	1.60	164.80	3 075
1 116	*B*	5	1.80	0.29	0.03	0.232	0.230	0.05336	1.48	152.44	2 856
1 117	*B*	6	1.80	0.18	0.03	0.232	0.230	0.05336	1.59	164.77	3 087
									Average =		3 046

TABLE No. 3.—Resilience of Wrought Iron.—Effect of Increasing the Initial Fall of the Hammer.

Kind of wrought iron.	Initial fall of hammer, in inches. F	Number of tests made.	Resilience, in inch-pounds per square inch of section at nick. R_2
Tennessee common.....	2	3	2 524
	4	3	2 488
Tennessee charcoal.....	2.5	3	3 549
	4.5	3	3 748

Note.—All the specimens of each metal were of the same form and dimensions, and were cut from the same bar.

TABLE No. 4.—NICKED IRON AND STEEL BARS.—RESILIENCE PER SQUARE INCH.

Bars nicked as shown in Fig. 8.

Metal.	Lot No.	Number of Melt.	Number of tests made.	Thickness of plate, in inches, t.	Resilience, in inch-pounds per square inch of section at nick, R_2.	Ultimate tensile strength, in pounds per square inch.*	Percentage of elongation in 8 ins.*
Iron, Norway.....	2	.....	8	0.25	7 535	41 500	28.2
" Tenn. common .	3	A	6	0.25	2 506	55 000	21.2
" " charcoal .	4	A	6	0.25	3 648	52 500	27.5
" " " .	4	B	6	0.25	2 885	.....	....
" " common.	3	B	6	0.25	2 115	.....	....
" common	5	A	6	0.25	2 176	50 700	14.2
" "	6	B	6	0.31	3 200	50 000	23.1
" "	6	C	6	0.37	2 140	52 500	18.1
" "	6	D	6	0.44	1 640	57 200	15.6
" "	6	E	6	0.50	3 500	54 800	19.6
" "	6	F	6	0.56	2 050	54 800	18.4
" "	6	G	6	0.62	2 080	52 800	19.0
Soft steel (plates)....	6	743	2	0.32	7 657	53 750	30.3
" " "	6	743	2	0.44	7 600	54 800	29.6
" " "	6	749	2	0.44	9 028	57 000	31.5
" " "	6	757	2	0.32	7 645	56 900	30.5
" " "	6	757	2	0.44	6 100	55 600	27.7
" " (angles)...	11	A	6	0.32	7 290	62 200	31.0
" " "	11	B	6	0.37	1 550	54 500	31.5
" " "	11	C	6	0.50	2 950	64 800	29.1

* The values in these columns are the result of one test only.

Table No. 3 contains the results of a few experiments made to show the effect of increasing the initial fall of the hammer. Apparently the effect is small compared with the variations in the metal.

Table No. 4 gives the values of resilience found with a number of samples of wrought iron and soft steel. In the last two columns are given for comparison the strength and ductility of each melt as shown by the ordinary tensile tests with the load applied gradually. The thickness t is in all cases the thickness of the plate or bar as rolled.

The steel of Lot No. 6 was basic open-hearth steel. The specimens were cut from large plates. The tests showed very uniform material. The steel of Lot No. 11 was said to be medium steel, but the tensile tests indicated that it should be graded as soft steel. These specimens were cut from angles about 3 by 3 ins. in size. These tests showed a lack of uniformity in the material. All the wrought-iron specimens were cut from bar iron.

On examining the values given in Table No. 4 we note that the highest value obtained with wrought iron is 7 535 inch-pounds per square inch, with ¾-in. Norway iron. The lowest value obtained with

wrought iron of the same thickness is 2 115 inch-pounds. Taking all thicknesses of wrought iron we get a range of from 1 640 to 7 535.

The highest test of steel is over 9 000 inch-pounds and the lowest is 1 550 inch-pounds.

In Melt No. 743 we get practically the same results with metal of different thicknesses, while in Melt No. 757 the thicker metal makes a poorer showing. This may be due to the thicker plate having been finished in the rolls at too great a heat.

A study of all the values given in Table No. 4 indicates that the resilience per square inch is not proportional to the ultimate strength, nor to the proportionate elongation, nor to the product of the two. We note, too, that the proportionate range in value is greater for the resilience than for either ultimate strength or elongation.

Conclusion.

In conclusion we may review the work briefly. The tests given were made by a new method of breaking small specimens in tension by impact. We find that there are two important errors which may be said to be peculiar to the method of testing. We find the first of these, or the error due to inertia, theoretically determinate. We find that the second of these errors, or that due to the spring of the fork-bar, is determinate to the degree that the tensile strength of the specimen is known. The other errors in the method are those common to all impact tests. We find that the results obtained are determined by the form of the specimen, and are hence only comparative.

We find that the tests that have been made by this method indicate that the resilience or shock resistance of rolled steel cannot be predicted from the tensile strength and elongation.

The values obtained will of course have but little other meaning until they have been interpreted by experience and by further experiment. It is suggested, however, that in time, tests of this sort may become a valuable aid in judging and recording the quality of structural steel.

DISCUSSION.

Mr. Broomall. C. M. Broomall, Jun. Am. Soc. C. E. (by letter).—All impact testing machines necessarily give truly comparative results only under similar conditions of striking hammer, supports, foundations, etc. These conditions are not realized in practice, and the best that can be done is to so arrange matters that the greater portion of the energy of the blow will be absorbed by the specimen. If the proportion of energy absorbed by the specimen is very large, the error caused by the difference in the conditions of the apparatus will not affect materially the accuracy of the results. In the machine designed by the author it is believed that the energy absorbed by the specimen is more than 90% of the whole, so that the results given by it must be quite accurate.

It seems to the writer, however, that a machine might be so designed that the specimen would absorb practically all the energy of the blow. Whether or not the mechanical details would be too complicated is another question. The suggestion is, to make use of a differential method of measurement, and, instead of mounting the specimen upon rigid supports, to mount it upon another pendulum initially at rest. If, then, after the impact, the rise of both pendulums be measured, the data are obtained from which to calculate the energy absorbed. This energy must have been entirely spent upon the specimen and its clamps. As these clamps are attached to the second pendulum, their weight must be added to it. The only energy not spent upon the specimen will be that absorbed in deflecting and producing angular motion of the clamps.

This differential method virtually amounts to suspending Mr. Russell's whole machine as a pendulum from the same axis as the striking pendulum, and so adjusting matters that the centers of percussion of the two pendulums are coincident with the point of impact.

Mr. Russell. S. Bent Russell, M. Am. Soc. C. E. (by letter).—Since the paper was presented to the Society some further experiments have been made by the writer, and it is thought to be in order to present them briefly in closing this discussion. The experiments given in the paper were all made on the first impact machine, which has a hammer or pendulum weighing 103 lbs. The second machine, lately completed, has a hammer weighing 203 lbs.

In order to throw more light on the errors of this method of testing, a series of experiments was made, testing the same metal with the two machines.

Table No. 5 gives the results of these comparative tests. All the specimens of each lot were cut from the same bar. The fork-bar used weighed 3.86 lbs. The heavier hammer gives a higher result in one

Mr. Russell.

case and a lower result in the other. This would indicate that increasing the weight of the hammer has no material effect on the result.

TABLE No. 5.—Effect of Increasing Weight of Hammer.

Metal, Wrought iron.	Lot No.	Resilience by 103-lb. hammer.	Resilience by 203-lb. hammer.
Tennessee charcoal.....	4	3 170	2 921
" common.....	3	2 169	2 325

Note.—Resilience is given in inch-pounds per square inch by tensile test. Test bar as shown in Fig. 8. Each value is the mean of six tests.

Similar comparative tests were made with cast-iron bars, broken transversely. All these tests showed a higher value of resilience for the 103-lb. hammer. The difference ran about 10%, indicating that the error of the machine is less with the heavier hammer.

Table No. 6 gives the results of tests made with the new machine since the paper was presented. The values may be compared with those of Table No. 4.

TABLE No. 6.—Nicked Steel Bars. Resilience Per Square Inch.

Bars nicked as shown in Fig. 8. Tested with 200-lb. hammer.

Metal.	Lot number.	Number of tests made.	Thickness of plate in inches, t.	Resilience in inch-pounds per square inch of section at nick, R_2.	Ultimate tensile strength in pounds per square inch.	Percentage of elongation in 8 ins.	Remarks.
Medium steel.....	14	6	0.25	7 100	*61 240	24.0	6-in. channel, flange.
					§60 240	30.7	
Soft "	16	6	0.44	1 660	60 320	27.5	Angle bar of 60-lb. rail.
" "	18	6	0.50	2 800	53 980	31.1	Planed from 6 x ¾-in. bars.
" "	17	6	0.38	6 270	56 830	28.4	
High "	15	5	0.44	2 140	110 530	11.6	60-lb. steel rail.
Cast "	8	5	0.38	1 808	73 380	7.0	.
" "	9	6	0.36	4 120	.....	....	

* Mill test, web.
§ Washington University test, flange.

The test of Lot No. 15 is of especial interest, showing that a high-grade steel may show an ultimate resilience, or resistance to shock, equal to that of a good grade of wrought iron. All these tests were made with a fork-bar weighing 3.86 lbs.

Now, in regard to the point raised by Mr. Broomall, it would seem to the writer that if such a machine as he describes were built, we would still fail in having all the energy of the blow absorbed by the specimen. In the case of tensile tests we would still have energy ab-

sorbed by the spring of the fork-bar, and on account of the inertia of the fork-bar. In transverse tests we would have energy disappear on account of the inertia of the specimen bar. The only losses which might be avoided by this device would be those due to the yielding of the fixed points of support. Now, from observations made upon the rigidity of these supports, it is believed that such errors may be made inconsiderable, in a properly designed machine, for all ordinary tests. Mr. Russell.

It might be profitable, however, to build such a machine as Mr. Broomall suggests, in order to determine more definitely and positively what energy is lost by yielding supports.

Vol. XLIII. JUNE, 1900.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

No. 866.

COMPARISON OF WEIGHTS OF A THREE-HINGED AND A TWO-HINGED SPANDREL-BRACED PARABOLIC ARCH.

By C. W. HUDSON, Assoc. M. Am. Soc. C. E.

PRESENTED SEPTEMBER 20TH, 1899.

WITH DISCUSSION.

In June, 1896, the author had occasion to calculate the stresses, sections and weight of a three-hinged spandrel-braced parabolic arch, the outline and main dimensions of which are shown in Fig. 1. Shortly after finishing these calculations he determined to make the corresponding calculations for an arch of the same outline having two hinges, in order to determine their exact relative economy as regards weight of metal. It was not until recently, however, that the second calculation was completed. As it would seem that these results might be of some interest to members of this Society, they are here given, together with a brief exposition of the method used in calculating the stresses in the two-hinged arch.

These arches were designed for a live load of 2 160 lbs. per lineal foot or 36 000 lbs. per panel of the arch, and a dead load of 2 880 lbs. per lineal foot or 48 000 lbs. per panel of the arch. This unusually large dead load is due to a heavy asphalt floor carried by buckle plates.

No metal less than $\frac{5}{16}$ in. in thickness was used for either case; and only two sizes of channels—12 and 15 ins.—were used for the members other than the arch ring. By introducing another shape, say 10-in. channels, a small saving of weight could have been made in both cases, but the appearance of the arch is better on account of the greater uniformity, and the construction is cheapened for the same reason.

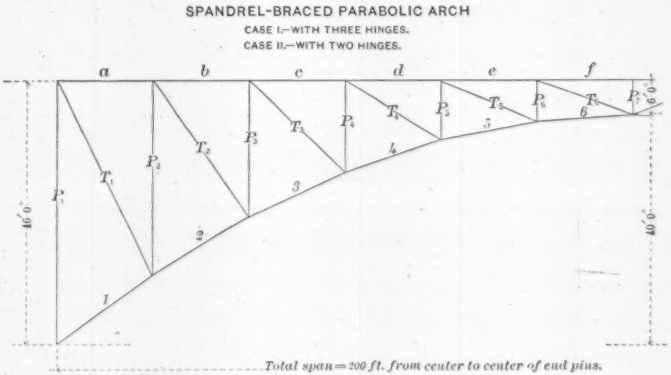


FIG. 1.

The unit stresses used in proportioning the members were:

For tension

Live-load stresses, 11 000 lbs. per square inch.

Dead-load " 22 000 " "

For compression

In arch ring and top chord

Live-load stresses, $12\,000 - 55\frac{l}{r}$ lbs. per square inch.

Dead-load " $24\,000 - 110\frac{l}{r}$ " "

In web members

Live-load stresses, $11\,000 - 50\frac{l}{r}$ lbs. per square inch.

Dead-load " $22\,000 - 100\frac{l}{r}$ " "

It would have been more consistent and better to have used for the arch rings and top chords also the compression formulas used for

web members, as the loading producing maximum stress in these members is partial, not covering the entire span, for almost all the members of both cases.

These unit stresses, as will be recognized, are those for medium steel, of Theodore Cooper's Specifications of 1896 for Highway Bridges.

The temperature stresses in the two-hinged arch were treated as dead-load stresses. Members subject to alternate stresses of tension and compression were proportioned to resist both kinds of stress, with $\frac{8}{16}$ of the smaller added to either.

TABLE No. 1.—Case I. Stresses, Sections and Weights for a Three-Hinged Arch Having a Span Length of 200 ft. from Center to Center of End Pins.

Member.	Live-load stresses.	Dead-load stresses.	Sections.	Sq. ins.	Weight in pounds.
P_1	+ 59 000 / − 77 000	24 000	2 15″ channels 100	= 19.6	
P_2	+ 50 400 / − 86 400	48 000	2 12″ " 89	= 17.4	
P_3	+ 37 900 / − 73 900	48 000	2 12″ " 65	= 12.7	
P_4	+ 24 700 / − 60 700	48 000	2 12″ " 65	= 12.7	14 230
P_5	+ 11 700 / − 47 700	48 000	2 12″ " 65	= 12.7	
P_6	+ 30 600 / − 66 600	48 000	2 12″ " 65	= 12.7	
P_7	− 36 000	48 000	2 12″ " 65	= 12.7	
T_1	± 65 800	.....	2 12″ " 100	= 19.6	
T_2	± 61 600	.....	2 12″ " 80	= 15.7	
T_3	± 54 900	.....	2 12″ " 65	= 12.7	
T_4	± 51 100	.....	2 12″ " 65	= 12.7	15 560
T_5	± 49 600	.....	2 12″ " 65	= 12.7	
T_6	± 130 800	.....	2 12″ " 138	= 27.1	
a	± 29 200	.....	2 15″ " 96	= 18.8	
b	± 64 500	.....	2 15″ " 96	= 18.8	
c	± 103 100	.....	2 15″ " 96	= 18.8	
d	± 134 300	.....	2 15″ " 124	= 24.3	14 230
e	± 122 800	.....	2 15″ " 124	= 24.3	
f	.....	.....	2 15″ " 96	= 18.8	
1	− 334 800	446 400	4 Ls. 4 × 4 × 48 / 2 pls. 24 × 11/16	= 51.8	
2	+ 7 100 / − 321 900	419 800	4 Ls. 4 × 4 × 44 / 2 pls. 24 × 11/16	= 50.3	
3	+ 21 600 / − 319 600	397 300	4 Ls. 4 × 4 × 48 / 2 pls. 24 × 5/8	= 48.8	
4	+ 41 500 / − 326 200	379 500	4 Ls. 4 × 4 × 44 / 2 pls. 24 × 5/8	= 47.3	36 180
5	+ 53 700 / − 329 200	367 100	4 Ls. 4 × 4 × 44 / 2 pls. 24 × 5/8	= 47.3	
6	+ 10 600 / − 281 200	360 800	4 Ls. 4 × 4 × 44 / 2 pls. 24 × 7/16	= 44.3	
			Details (from shop drawings)....		34 800
			Total weight =		115 000

Case II.—The stresses for the two-hinged arch are not statically determinate, but may be obtained from the elastic properties of the arch.

The vertical components of the reactions for any load are the same as the reactions for a simple truss of the same span. The horizontal component of the reactions for any load can be found from the following formula:*

$$H = P \frac{\delta}{\delta'}$$

In which

H = Horizontal thrust produced by P;

P = Load at any point;

δ = Vertical deflection of the loaded point, due to a horizontal force of unity acting at the free hinge—one hinge assumed fixed and the other free—;

δ' = Horizontal displacement of the free hinge, due to a horizontal force of unity acting at the free hinge.

Fig. 2 will make the notation of this formula more clear than the definitions alone could.

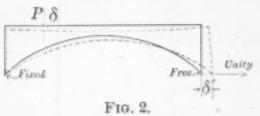


FIG. 2.

For simplicity of comparison the results are arranged as compactly as possible in Table No. 2.

The first column gives the marks designating the members (see Fig. 1). The second column shows the stresses due to a horizontal force of unity acting at one of the hinges. These stresses were very carefully figured analytically, and then checked graphically by means of the diagram, Fig. 3. From these stresses the changes in length λ of

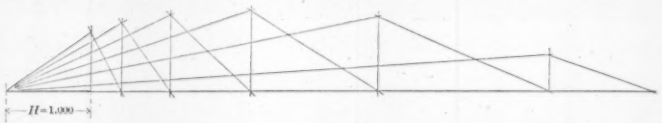


FIG. 3.

the various members were computed from the formula $\lambda = \frac{T L}{E A}$. In determining these values of λ, which are given in the third column, L was taken in feet, A, at present unknown, was taken equal to unity, and E was taken at 29 instead of 29 000 000, in order to give values of λ sufficiently large to allow them to be accurately plotted. These pre-

* "Roofs and Bridges," Part IV, Merriman and Jacoby.

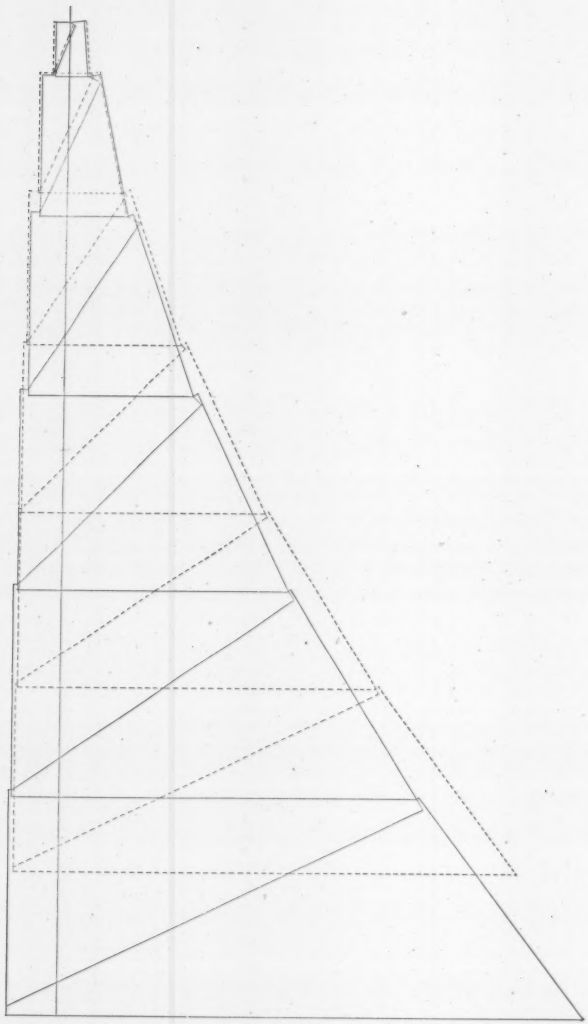


FIG. 4.

liminary values of λ are 1 000 000 $\times$ A times their true value. Using these values of λ the displacement diagram shown in dotted lines in Fig. 4 was constructed. The full lines in this figure give the final displacement diagram, and from this diagram the ratio of δ to δ' was found to have the following values, beginning with the panel point at the end of the arch: 0.003, 0.203, 0.401, 0.590, 0.764, 0.904 and 0.962.

The thrust due to a load at the end of the span being so small, only 0.003 of the load, it is neglected in both the preliminary and final calculations.

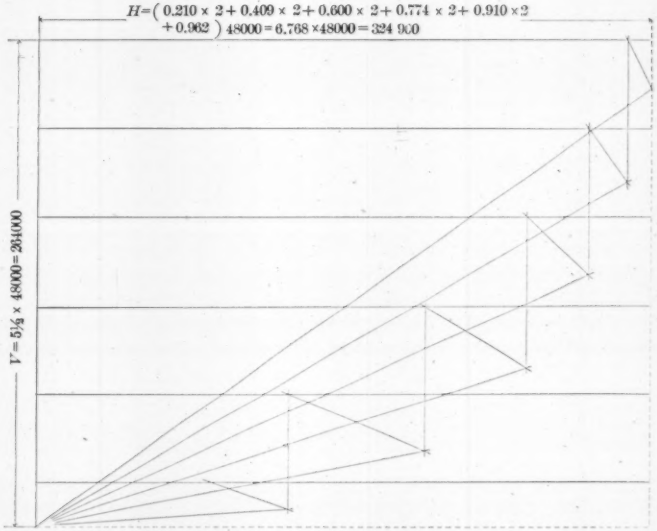


FIG. 5.

For full loading the preliminary value of the horizontal thrust is 6.686 $\times$ 48 000 lbs.; from this, by means of a diagram similar to Fig. 5, (Fig. 5 being the final dead-load stress diagram), the preliminary dead-load stresses are obtained. These stresses are given in the sixth column of the table.

Before finding the preliminary live-load stresses it will be well to find the stresses in the arch due to a vertical reaction of unity, supposed to be applied in this case at the left hinge. The stresses due to this load are given in the fifth column of the table. They were com-

TABLE No. 2.—(*Continued*).

a ..	+ 0.362	+ 0.208	+ 0.011	— 0.493	— 14 500	— 12 700	+ 20 100 — 30 700	+ 20 800 — 30 200	3 600	2 15″ " 96 = 18.8	2 15″ " 96 = 18.8		
b ..	+ 0.935	+ 0.537	+ 0.029	— 7.009 — 1.402	— 36 500	— 33 200	+ 41 400 — 68 700	+ 43 000 — 67 500	9 400	2 15″ " 96 = 18.8	2 15″ " 96 = 18.8		
c ..	+ 1.875	+ 1.078	+ 0.057	— 9.375 — 3.125	— 74 500	— 66 300	+ 61 000 — 115 500	+ 64 000 — 113 400	18 800	2 15″ " 96 = 18.8	2 15″ " 96 = 18.8		
d ..	+ 3.404	+ 1.956	+ 0.059	—12.766 — 6.383	— 134 100	— 120 900	+ 66 500 — 166 300	+ 71 500 — 166 900	34 000	2 15″ " 169 = 33.2	2 15″ " 159 = 31.2		
e ..	+ 5.469	+ 3.143	+ 0.095	—16.406 — 11.719	— 214 700	— 192 000	+ 36 100 — 196 300	+ 44 200 — 188 300	55 000	2 15″ " 169 = 33.2	2 15″ " 159 = 31.2		
f ..	+ 6.667	+ 3.831	+ 0.107	—16.667	— 260 900	— 232 800	— 195 400	+ 800 — 176 400	66 700	2 15″ " 183 = 35.9	2 15″ " 166 = 32.6	17 890	
1 ..	— 1.240	— 0.884	— 0.018	+ 0.000	— 398 000	— 403 000	— 298 500	— 302 100	12 400	{4 Ls. 4 × 4 × 51 {2 pls. 20 × $\frac{3}{4}$ } = 50.0	{4 Ls. 4 × 4 × 53 {2 pls. 20 × $\frac{3}{4}$ } = 50.8		
2 ..	— 1.588	— 1.064	— 0.023	+ 0.575	— 357 600	— 364 000	+ 7 400 — 275 700	+ 7 000 — 281 200	15 900	{4 Ls. 4 × 4 × 51 {2 pls. 20 × $\frac{3}{8}$ } = 45.0	{4 Ls. 4 × 4 × 53 {2 pls. 20 × $\frac{3}{8}$ } = 45.8		
3 ..	— 2.135	— 1.354	— 0.033	+ 7.735 + 1.547	— 313 200	— 321 800	+ 23 200 — 258 600	+ 22 000 — 260 200	21 400	{4 Ls. 4 × 4 × 41 {2 pls. 20 × $\frac{3}{8}$ } = 41.1	{4 Ls. 4 × 4 × 43 {2 pls. 20 × $\frac{3}{8}$ } = 41.9		
4 ..	— 3.031	— 1.896	— 0.047	+ 9.882 + 3.294	— 259 900	— 272 900	+ 47 600 — 243 500	+ 44 900 — 249 730	30 300	{4 Ls. 4 × 4 × 41 {2 pls. 20 × $\frac{9}{16}$ } = 38.6	{4 Ls. 4 × 4 × 43 {2 pls. 20 × $\frac{9}{16}$ } = 39.4		
5 ..	— 4.492	— 2.633	— 0.083	+13.019 + 6.509	— 190 600	— 209 100	+ 74 000 — 217 000	+ 68 300 — 224 700	44 900	{4 Ls. 4 × 4 × 29 {2 pls. 20 × $\frac{1}{2}$ } = 31.4	{4 Ls. 4 × 4 × 29 {2 pls. 20 × $\frac{9}{16}$ } = 33.9		
6 ..	— 6.483	— 3.735	— 0.117	+16.442 +11.745	— 107 000	— 133 400	+ 72 000 — 152 500	+ 62 400 — 162 100	64 800	{4 Ls. 4 × 4 × 29 {2 pls. 20 × $\frac{1}{2}$ } = 31.4	{4 Ls. 4 × 4 × 29 {2 pls. 20 × $\frac{1}{2}$ } = 31.4	30 560	
											Details (estimated)....	31 900	
											Total weight =	109 000	

*The Temperature Stresses in the Final Calculations were found to be the same as in the Preliminary Calculations.

TABLE No. 2.—Case II.—Stresses, Sections, Weights, etc., for a Two-Hinged Arch, Having a Span Length of 200 ft. from Center to Center of End Pins.

Truss Members.	Stresses due to H.	λ		Stresses due to V.	Dead-Load Stresses.		Live-Load Stresses.		Temperature Stresses.	Sections.		Weights of final sections in pounds.
		Prelim.	Final.		Prelim.	Final.	Prelim.	Final.	Preliminary and Final.*	Preliminary.	Final.	
P_1..	+ 0.733	+ 1.162	+ 0.062	— 1.000	— 53 000	— 49 800	+ 40 700 — 80 300	+ 42 000 — 79 400	± 7 300	2 15″ channels 96 = 18.8 Sq. Ins.	2 15″ channels 96 = 18.8 Sq. Ins.	
P_2..	+ 0.817	+ 0.952	+ 0.051	— 2.257 — 1.296	— 80 000	— 76 900	+ 30 800 — 90 800	+ 32 000 — 89 600	8 200	2 15″ “ 96 = 18.8	2 15″ “ 96 = 18.8	
P_3..	+ 0.903	+ 0.740	+ 0.044	— 2.271 — 1.654	— 85 200	— 79 300	+ 20 100 — 82 500	+ 21 400 — 81 100	9 000	2 12″ “ 85 = 16.7	2 12″ “ 82 = 16.1	
P_4..	+ 0.958	+ 0.529	+ 0.039	— 2.125 — 2.042	— 86 000	— 82 000	+ 8 500 — 72 700	+ 10 500 — 71 900	9 600	2 12″ “ 70 = 13.7	2 12″ “ 69 = 13.5	
P_5..	+ 0.881	+ 0.317	+ 0.025	— 1.553 — 2.277	— 82 700	— 79 000	+ 9 900 — 71 600	+ 10 700 — 69 900	8 800	2 12″ “ 65 = 12.7	2 12″ “ 65 = 12.7	
P_6..	+ 0.431	+ 0.106	+ 0.008	— 0.694 — 1.781	— 64 600	— 62 800	+ 27 600 — 76 200	+ 28 100 — 75 500	4 300	2 12″ “ 65 = 12.7	2 12″ “ 65 = 12.7	
P_7..	.....	.....	.....	.....	— 48 000	— 48 000	— 36 000	— 36 000	.....	2 12″ “ 65 = 12.7	2 12″ “ 65 = 12.7	15 220
T_1..	— 0.818	— 1.062	— 0.084	+ 1.115	+ 32 200	+ 28 800	— 45 500 + 69 500	— 46 900 + 68 400	8 200	2 12″ “ 65 = 12.7	2 12″ “ 65 = 12.7	
T_2..	— 0.998	— 0.999	— 0.079	+ 2.756 + 1.583	+ 38 900	+ 35 300	— 37 000 + 66 000	— 39 100 + 65 400	10 000	2 12″ “ 65 = 12.7	2 12″ “ 65 = 12.7	
T_3..	— 1.304	— 1.039	— 0 082	+ 3.279 + 2.389	+ 53 000	+ 45 500	— 29 000 + 67 200	— 30 900 + 65 100	13 000	2 12″ “ 65 = 12.7	2 12″ “ 65 = 12.7	
T_4..	— 1.805	— 1.224	— 0.097	+ 4.002 + 3.845	+ 70 900	+ 63 600	— 19 000 + 72 000	— 22 800 + 70 300	18 000	2 12″ “ 65 = 12.7	2 12″ “ 65 = 12.7	
T_5..	— 2.245	— 1.403	— 0.095	+ 3.958 + 5.801	+ 88 000	+ 78 300	— 39 400 + 105 600	— 42 400 + 101 400	22 500	2 12″ “ 79 = 15.5	2 12″ “ 75 = 14.7	
T_6..	— 1.273	— 0.778	— 0.033	+ 0.277 + 5.259	+ 49 300	+ 43 000	— 118 700 + 155 700	— 122 800 + 154 200	12 700	2 12″ “ 120 = 23.5	2 12″ “ 121 = 23.7	13 430

puted analytically and checked by means of the diagram, Fig. 6. The upper figures in the fifth column, where two sets are given, are the stresses for the corresponding member in the right half of the arch.

To determine the greatest tensile and compressive live-load stresses to which any member of the arch is subjected, take *5* of the arch ring for example, and arrange the computations as follows:

$-H$		$+V$	
4.492×0.203		$13.019 \times \frac{1}{12}$	
$\times 0.401$	1.958	$\times \frac{2}{12}$	$\frac{5}{6} = 10.849$
$\times 0.590$		$\times \frac{3}{12}$	
$\times 0.764$	3.432	$\times \frac{4}{12}$	4.340
$\times 0.904$	4.061	$6.509 \times \frac{7}{12}$	3.797
$\times 0.962$		$\times \frac{6}{12}$	
$\times 0.904$		$\times \frac{5}{12}$	
$\times 0.764$	4.728	$\times \frac{4}{12}$	$2\frac{1}{3} = 15.208$
$\times 0.590$		$\times \frac{3}{12}$	
$\times 0.401$		$\times \frac{2}{12}$	
$\times 0.203$		$\times \frac{1}{12}$	

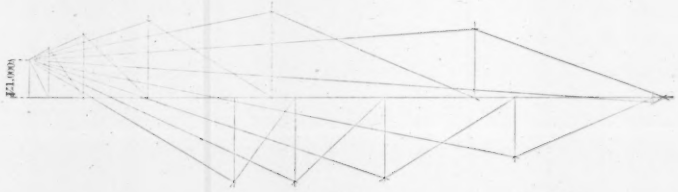


FIG. 6.

The value of H and V being those given in the second and fifth columns of Table No. 2 opposite *5*, by inspection it can be seen that loads on the second, third, fourth and fifth panel points produce tension, and loads on the other points produce compression in the member *5*. The computation of the stresses is now very simple, and we have:

For tension $(10.849 - 4.492 \times 1.958)\ 36\,000 = 2.054 \times 36\,000$
$= 74\,000.$

For compression $(4.728 \times 4.492 - 15.208)\ 36\,000 = 6.030 \times 36\,000$
$= 217\,000.$

The other live-load stresses were determined in this manner, and the computations were equally simple.

Having determined the preliminary live and dead-load stresses, it now remains to determine the preliminary temperature stresses. From the displacement diagram, already constructed, the horizontal displacement of the hinge, due to a horizontal force of unity, is found to be $\frac{230.8}{1\,000\,000 \times a}$ ft. It will be assumed that the arch will be subjected to a range of temperature of 120° Fahr., or a variation of 60° from the mean. This is a less variation than is usually assumed, but as the metal of the arch under consideration is to be protected from the direct rays of the sun by a highway floor, it is probably enough. The change in length of the arch due to a variation of 60° Fahr., taking the coefficient of expansion as 0.0000065 per degree Fahr., is 0.078 ft. Dividing this change of length due to temperature by the change in length due to a thrust of 1 lb., we get $\frac{78 \times a \times 10\,000\,000}{1\,000 \times 2\,308}$ lbs. as the value of the temperature thrust. The value of a in this expression for the value of the temperature thrust is at present unknown; the average influence of the areas of all the members of the arch is very nearly represented by the influence of the area of a member of the top chord near the center of the arch. In this case the approximate value of the area of the member e, as determined from the preliminary live and dead-load stresses, is used as the value of a in the expression for the temperature thrust. This approximate determination of the area of e gives about 31 sq. ins., which gives for the preliminary temperature thrust $\frac{78 \times 31 \times 10\,000\,000}{1\,000 \times 2\,308} = 10\,000$ lbs. The preliminary temperature stresses are now determined by multiplying the stresses in the second column of the table by 10 000.

From these preliminary dead-load, live-load and temperature stresses the preliminary sections were determined.

Using these new areas, instead of unity used in the preliminary calculation, the final calculations of stresses are made in exactly the same manner as the preliminary calculations. It is found that the preliminary temperature stresses need not be changed for the final determination of the sections. A final determination of the sections shows only small changes from the preliminary. In most of the members there is no change, and the greatest change amounts to only 6%, and as the preliminary determination for this case was on the safe side it seems that the final calculation was hardly necessary. Where

an estimate only is required, the preliminary calculation would certainly be ample.

The calculations of weight give:

For Case I	115 000 lbs.
For Case II	109 000 "

This gives a saving in weight of 5½% in favor of the two-hinged arch. A variation of 75° in temperature would have lessened the difference considerably, but the two-hinged arch would still have been lighter.

While it is hardly allowable to draw general conclusions from the consideration of a special case, it can be said that where an arch of this form (spandrel-braced) is suitable, the two-hinged arch is lighter than the three-hinged arch; it is also cheaper to construct, as there is no center pin and there are no adjustable members at the center of the arch. The floor system of the two-hinged arch would also be more simple than that of the three-hinged arch, for the great range of height of the center-pin of the three-hinged arch, due to temperature and live-load stresses, makes a troublesome break in the floor system at this point.

When spandrel-braced arches are used in series, supported on intermediate masonry piers, the two-hinged arch has the advantage of having less horizontal thrust, and therefore requiring smaller piers than the three-hinged arch. Great care must be taken in the construction of the masonry for the two-hinged arch, in order that it may not be subject to even slight settlement or displacement; but, taking this extra work into account, it is believed that the masonry for a series of two-hinged arches will cost less than the masonry for a series of three-hinged arches.

DISCUSSION.

HENRY S. JACOBY, Assoc. Am. Soc. C. E. (by letter).—Since the adoption of the design of the spandrel-braced steel railway arch over the Niagara River, and the beginning of its construction, the writer has taken a renewed and more definite interest in the comparative study of different types of metallic arches. At his suggestion, two graduate students in the College of Civil Engineering of Cornell University last year made the computations necessary to determine the relative weight of three-hinged and two-hinged spandrel-braced arches having the same general dimensions and loading as the Niagara Railway Arch. Mr. George G. Smith, Jr., made the design for the two-hinged arch, while Charles C. More, Jun. Am. Soc. C. E., made the design for the three-hinged arch as well as that of a combination-type arch, which will be described in the latter part of this discussion. Mr. Jacoby.

The general dimensions, loading and specified unit stresses are given in the paper on "The Niagara Railway Arch."*

In finding the stresses in the trusses, however, two excess panel loads were substituted for the excess of the two locomotives above the corresponding weight of the train.

The Two-Hinged Arch.—In view of the statement made on page 137 of that paper, the given sectional areas of the members of the arch were not used except to find the stresses adopted in re-designing the sections. After this an additional revision was made, in order to see whether the final sections should generally depend upon the second or the third series of stresses, the first series being the preliminary ones which depend upon an assumption of equal sectional areas for all the members.

It was found that the required areas of the upper chord were increased from 0.5 to 1.1%, except those of U_7 and U_8 in Fig. 7, in which the increase was 2.9 and 3.3%, respectively. The areas of L_6 and L_7 were increased 0.5%, while those of the remaining lower-chord members were diminished from 0.4 to 1.3 per cent. The areas of the diagonals were increased from 0.2 to 1.8%, except that of D_5, in which the increase was 6.8 per cent. In the verticals, the increase of section varied from 1.2 to 3.5%, except

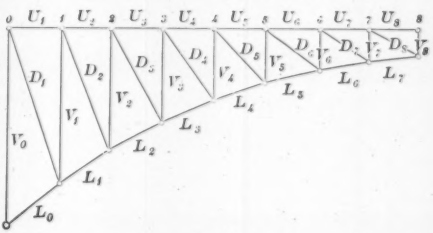


FIG. 7.

* By R. S. Buck, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. xl, p. 125.

Mr. Jacoby. for V_7, in which it was 6.8 per cent. Since most of these differences were covered by the excess of the sections adopted over those required, only a few members were affected by this revision.

These changes in section are relatively much smaller than one is led to expect from the corresponding differences in the stresses, the result being due to the influence of the reverse stresses in the design. The temperature stresses were derived for a range extending from 75° above to 75° below the standard.

The above results indicate that the desirability of making the second revision depends largely upon the magnitude of the structure, while it is evidently affected to some extent by the form and proportion of the trusses.

The Three-Hinged Arch.—The dead-load stresses for the three-hinged arch were first found by using the same panel loads as those which had been determined from the design of the two-hinged arch. After this preliminary design was completed the dead panel loads were computed and found to exceed the previous values by percentages varying from 0.8 near the middle to 3.6 near the ends of the truss, while the middle panel load was 0.3% less. The average excess is 2.32 per cent. The dead-load stresses were revised accordingly, and the following differences were found, expressed as percentages of the sum of the dead, live and 0.8 times the reverse stresses. For the upper chord, 0.8 — 1.9; for the lower chord, 1.3 — 2.0; for the diagonals, 0.3 — 2.0; and for the verticals, 0.2 — 2.4. All the stresses were increased, except those in D_1, V_6, V_7 and V_8.

These changes, however, affected the sectional areas of only four members, the rest being covered by the small excesses in area made necessary by the make-up of the sections.

Comparative Weights.—Excluding the connecting plates, rivet heads and some minor details, all of which may be reasonably assumed to vary in the same ratio as the members of the respective trusses, it was found that the trusses of the three-hinged arch were 0.84% lighter than those of the two-hinged arch. The material is distributed as shown in Table No. 3.

TABLE No. 3.

	Upper chords.	Lower chords.	Diagonals.	Verticals.
Two-hinged arch.....	15.6%	36.7%	22.2%	25.5%
Three-hinged arch.....	12.9%	43.4%	20.1%	23.6%

It will be observed that in the three-hinged arch a considerably larger proportion of the material is contained in the lower chord, while the other classes of members contain less material than the two-hinged

arch. Only four members in these classes, U_3, U_4, D_1 and D_2 are larger in the three-hinged arch. It may be of interest to note that the range of stress in the lower chord of the three-hinged arch is 6.5% less, while the maximum stress (excluding the wind) averages 17.0% more, and the live-load stress 26.8% more, than in the two-hinged arch. Mr. Jacoby.

In order that the influence of the crown hinge might be obscured as little as possible by extraneous conditions, the designs of the corresponding members in the two-hinged and three-hinged arches were made as nearly alike in every respect as the required sectional areas would permit, the differences in the composition or make-up of the sections from that used in the construction of the Niagara Railway Arch, being reduced to a minimum. Since the distance from back to back of the angles in the chords was made the same in all cases, the additional material required in the lower chord of the three-hinged arch had generally to be placed on the inside and in the web plates, the result being to reduce the squares of the radii of gyration by an average of 4 per cent. This element places the three-hinged arch at a slight disadvantage.

Comparison with the Arch of 200-Ft. Span.—The corresponding distribution of material in the trusses described by the author is given in Table No. 4.

TABLE No. 4.

	Upper chord.	Lower chord.	Diagonals.	Verticals.
Two-hinged arch	23.2%	39.6%	17.4%	19.7%
Three-hinged arch	17.7%	45.1%	19.4%	17.7%

The web members are relatively lighter than in Table No. 3, while the chords are heavier, notably the upper chord. Some of this disparity may be accounted for by the difference in the working stresses, as well as the differences in the relative loading and in the proportions of the trusses. The uniform live load in the Niagara Arch is 2.3 times as great, while the dead load is only 1.8 times as great per linear foot as in this arch. Again, the span is 2.75 times as great, while the depth at the crown is 3.33 times as great. The ratio of the rise of the lower chord to the span is very nearly the same. The trusses of the Niagara Arch are about 10 times as heavy as those of the smaller arch. The range of temperature was assumed as 25% greater. The differences between the required sectional areas and those adopted in the design are relatively much larger in the smaller arch on account of the smaller sections, and the greater influence of the limitations imposed by the minimum thickness of metal allowed.

The different conditions just stated likewise affect the relative

Mr. Jacoby. weights of the trusses of the two types of arches, but it would require additional investigation to determine the magnitude of this effect.

One other element should be mentioned, namely, the effect of the wind stresses upon the truss members. For the comparative designs of the Niagara Arch it was assumed that no section should be increased in area unless the wind stress exceeded 40% of the sum of the dead, live and temperature stresses. It was found that this required an increase in only two chord members near the middle in the two-hinged arch, and one in the three-hinged arch.

It may, perhaps, be well to call attention to the fact that if in the smaller arch described in the paper the weight of the details be excluded, the difference in favor of the two-hinged arch is reduced to 3.9%, while if the details are included and made the same in both cases the differences will be only 2.7 per cent.

A Combination-Type Arch.—Any slight yielding in the foundations of a two-hinged arch materially changes the stresses, especially near the crown, while inaccuracies in the construction, in locating the end hinges, or in adjustment in closing the arch, have a similar effect. In view of these facts, it appeared to the writer that it would be desirable to erect an arch with three hinges, and, after completion, when all the dead load is in place, to transform it into a two-hinged arch by connecting, at the standard temperature, the upper chord at the center and by riveting the connection at the crown hinge. Such an arrangement eliminates the effect of any inaccuracies in construction or erection, as well as those of the initial set of the abutments, due to the imposed loads. The resulting stresses are a combination of the dead-load stresses of a three-hinged arch with the live-load and temperature stresses of a two-hinged arch.

It seemed worth while to determine how the weight of such an arch would compare with the other two types, and whether it would possess any other merit than that of realizing more perfectly in construction the conditions which are assumed in its design than is done by the two-hinged arch.

In the first determination of the sectional areas the stresses obtained in the preceding designs were used, and then a revision was made with the aid of a new set of live and temperature stresses. The temperature stresses were $10\frac{3}{4}\%$ greater than before. In this revision the required sectional areas were increased, with three exceptions, by from 0.4 to 8.2%, the difference being below 2% in all but a few members. One-half of the areas at first adopted were increased by amounts varying from 1.0 to 3.8%, except one which had to be enlarged 5.9 per cent.

The weight of this truss, exclusive of the connecting details, was found to be 4.2% greater than that of the two-hinged arch. This excess in weight is due, mainly, to the influence of the larger reverse

stresses in many of the members. The distribution of weight is as follows: Upper chords, 16.5%; lower chords, 37.2%; diagonals, 22.3%; and verticals, 24.0 per cent. Mr. Jacoby.

Comparing the various classes of members with those of the two-hinged arch, the results are: Upper chords, 9.7% heavier; lower chords, 5.8% heavier; diagonals, 4.8% heavier; and verticals, 1.9% lighter. With the three-hinged arch as a basis of comparison, the upper chord is 34.0% heavier; the lower chord, 9.8% lighter; the diagonals, 16.3% heavier; and the verticals, 7.0% heavier.

The distribution of the material in this combination-type of arch is such that it forms a stiffer structure than the two-hinged arch, its static deflection at the center under full live-load being nearly 10% less.

The writer wishes to express his appreciation of the great care exercised by Messrs. More and Smith in making the computations and graphic constructions involved in these designs, and from which this discussion has been prepared.

In conclusion, a note may be added in the interest of diminishing the labor required to find the stresses. If the stresses in the upper chord due to $H = 1$ and $V = 1$ be computed and laid off on the horizontal lines in Figs. 3 and 6, respectively, experience shows that the rest of the stresses may be determined graphically with a precision that answers fully all the requirements of design, it being understood that a suitable scale is adopted. This procedure will save the tedious computations for the lengths of the lever arms of the remaining truss members.

C. W. Hudson, Assoc. M. Am. Soc. C. E. (by letter).—It is generally fair to assume that the weights of details for trusses of the same type, and not greatly varying spans, are a certain percentage of the weights of the main members, but it is not to be assumed that the same percentage holds between different types of trusses. Mr. Hudson.

In the case of the three-hinged spandrel-braced arch compared with the two-hinged arch, it is evident that the details of the three-hinged arch will be heavier than those of the two-hinged arch by at least the following: First, the weight of the center pin; second, the weight of part of the pin plates bearing on the center pin; third, the weight of additional details, for center top chord and center vertical post, to give proper adjustment for these members for the great range of height of the center pin; and fourth, for the case of the 200-ft. arch spans compared, there was a great saving in details in favor of the two-hinged arch, due to the fact that the arch ring of the three-hinged arch was of greater dimensions, thereby requiring larger gusset plates for connection to web members, also requiring larger and heavier battens and lattice throughout the arch, as the members of the arch other than the ring had to be made wider than necessary in order to take the arch-ring gussets. These heavier battens, lattice, and gussets of the arch

Mr. Hudson. ring could have been avoided by taking a ring of the same size as for the two-hinged arch; but then there would have been a loss of about 1 500 lbs. in the main material of the arch ring, due to the greater value of $\frac{l}{r}$.

From this we see that the fourth saving can be made, either in the details of the entire arch or in the main material of the arch ring. This is due to the fact that there is not such a great difference between the sections of the arch ring and the sections of the other members of the arch for the two-hinged as for the three-hinged arch.

Vol. XLIII. JUNE. 1900.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

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No. 867.

THE GROINED ARCH AS A COVERING FOR RESERVOIRS AND SAND FILTERS: ITS STRENGTH AND VOLUME.

By LEONARD METCALF, Assoc. M. Am. Soc. C. E.

PRESENTED SEPTEMBER 6TH, 1899.

WITH DISCUSSION.

Some months ago the writer had occasion to figure the volume of masonry in the elliptical groined arch roof covering the sand filter then under construction by William Wheeler, M. Am. Soc. C. E., for the Somersworth, N. H., Water-Works (see Plate II, Fig. 1), and deduced therefor a series of formulas giving the volume of masonry in its cylindrical, cloistered and groined elliptical arches. These formulas, which have been considerably amplified, and to which have been added a sketch of the development of the groined arch, a comparison of the relative volume of masonry in the several types of arches, and a brief description of a method for computing the strength of the groined arch, form the basis of this paper.

The results thus far obtained by the writer are presented at this time, not with the idea that they are in any sense complete, but in the hope that they may be of some service to engineers, and that they

may, perhaps, suggest to others new methods of analysis which may lead to the deduction of general formulas, now entirely wanting, for the dimensioning and comparison of the different types of arches.

Among the most common forms of masonry roofs in use are the following: *First*, the dome or spherical arch; *second*, the cloistered arch; *third*, the cylindrical arch, and *fourth*, the groined arch—all still further subdivided, according to the form of the generating curve into semi-circular, elliptical, segmental, parabolic, pointed or Gothic, multi-centered and various other forms.

Of these four general types of arches, the dome and the cylindrical or barrel arch are too well known to require description. The cloistered and the groined arch, however, should be carefully distinguished, the essential difference being that in the cloistered arch the vaulting springs from a series of springing lines and meets at the crown in a common point, the several vault intersections or arrises forming re-entrant angles in the masonry; whereas, in the groined arch, the vaulting springs from a series of springing points or pillars and meets at the crown in a series of intersecting lines (see Figs. 1 and 2 and photographs of Somersworth, N. H., and Ashland, Wis., filter plants, Plate II), the several vault intersections or arrises forming so-called groins or ribs, convex angles in the masonry.

It will thus be seen that the groined arch, even more than the dome, gives, naturally, effects of great spaciousness, simplicity and stability of form, well adapted to use in such engineering structures as covered reservoirs, etc., in which economy and stability of construction, and air space for convenience in operation and ventilation, are prime requisites, and in which the presence of supporting pillars is no serious objection.

The history of the groined and the cloistered arch is coeval with the development of the early ecclesiastical structures, and is of remote origin, probably dating back to the time of Roman supremacy; for with the rise of Roman power, we find arched or vaulted types of masonry construction supplanting the post and lintel form of the early Grecians, and passing subsequently through many modifications, tending always toward greater ornamentation and splendor.

Necessity was probably the occasion of the invention of the groined arch, for the earliest custom of the Romans, when building two intersecting arches, was to raise one arch above the other, in order to avoid

FIG. 1.—COVERED FILTER, SOMERSWORTH, N. H.

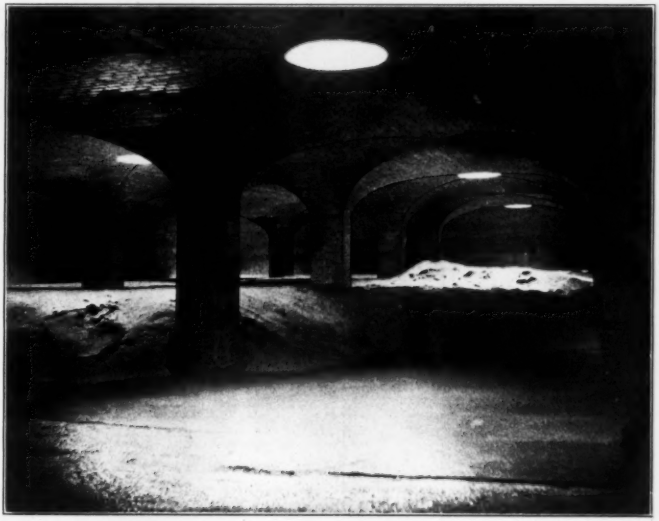


FIG. 2.—COVERED FILTER, ASHLAND, WIS.

their intersection, and the consequent formation of groins, a clumsy though satisfactory method where practicable. But situations soon arose in which this method was impracticable on account of lack of head room, and the groined arch was the inevitable result. Moreover, a common form of arrangement of columns and roof in the early public buildings of the Romans was, according to Choisy, that of a large central vaulted hall or church, flanked on either side by secondary vaulted chambers, which necessitated the use of groined arches.

The origin of the groined arch has thus been traced back to the second or third century of the Christian era, and many striking examples of its magnitude are known. Russell Sturgis, in his "European Architecture," and Auguste Choisy, in his admirable and comprehensive work on "L'Art de Bâtir chez les Romains," describe them at length, citing the great Tepidarium or Baths of Caracalla, built at Rome in 215 A. D., of 82 ft. span; the Baths of Diocletian at Rome, built in 290 A. D. and restored by Michael Angelo in the sixteenth century, and the Basilica of Maxentius and Constantine, of 86 ft. span, portions of which are still standing near the Forum at Rome. Yet, while these massive ancient structures, with all their difficulties of erection and construction, excite our interest and wonder, they could scarcely be cited as models of economical construction.

The first groined arches were characterized by their simplicity, the vaulting—generally semi-circular—being unornamented, the groins merely sharp arrises. Later, they furnished the chief decoration of the edifices in which they were built, and were richly carved and ornamented. With the increasing demand for greater unobstructed space and more striking architectural effects in the large baths and halls, the dimensions of the vaulting grew, as did the difficulty of adequately centering the structure for construction. This, the growing tendency to ornamentation of the groins and the introduction of the Gothic arch, probably suggested the use of ribs to re-enforce the groins or arrises, which ribs thus divided the vaults into sections and permitted the reduction of the thickness of the vaulting between them and the discarding of a portion of the heavy centering.

These ribs were at first very simple in section, being generally broad and thin, with rectangular off-sets, much as the architrave of a door; and, even up to the twelfth century, according to Gwilt, "were seldom moulded with more than a simple torus or some fillets." The

ribs sprung from the square piers supporting the four corners of the vaulted roof, the groins corresponding to the diagonals of the vault; but in the thirteenth, fourteenth and fifteenth centuries, the section of the supporting columns, the form of groining and number of ribs, grew more complex, and in following the changes and multiplication of the number of ribs we may trace the evolution of the fan-shaped, the star and the many other forms of groining developed during those centuries, described at length and fully illustrated in Dr. G. U. Breymann's "Bau Constructions Lehre," page 151; in Auguste Choisy's "L'Art de Bâtir chez les Romains,"* and in Ware's "Tracts on Vaults and Bridges."

It might be noted, in concluding this brief outline of the early development of the groined arch, that the materials for the vaulting used by the Romans were stone or brick, backed with concrete or rubble, and the ribs were generally built of the same material, sometimes specially moulded, the rib joints, particularly in the smaller ribs, being laid either in cement mortar or with sheet lead, to distribute the pressure more uniformly over the joint. In later days wood and iron came to be used in place of masonry as the building material.

Turning now to a consideration of the adaptability of the groined arch as a roof covering for large reservoirs, sand filter beds or similar structures in which large spans are not required nor piers objectionable, the following points of advantage may be claimed for it in comparison with the cylindrical, the cloistered and the spherical or domed arch.

First.—Greater air space or spaciousness under the vaulting, which tends toward better circulation of air and consequent aeration of the water, and which, in the case of sand filter beds, gives more room for scraping and renewing the surface of the beds.

Second.—Economy of material.

Third.—Absence of necessity for lintel arches.

Fourth.—Equal or greater adaptability to lighting, ventilation and access from above.

Fifth.—Ease of construction and equal adaptability to concrete construction, except as compared with the cylindrical arch, for which

* Illustrations from Choisy's work are also to be found in a short paper on "Roman Construction," by G. W. Perry, published, since the preparation of this paper, in the *Journal* of the Association of Engineering Societies, October, 1898, Vol. xxi, No. 4.

the centers can be somewhat more easily and cheaply built, though the necessity for centering for the lintel arches in the latter would perhaps off-set this advantage of the cylindrical arch.

The groined arch is, of course, best adapted to roofing square or rectangular areas, for though circular areas can be covered in this way, the greater difficulty of forming the arch centers for the latter is apparent, as they have to be cut around the perimeter of the chamber on a spiral or other curve formed by the intersection of the roofing and the side walls.

The superiority of the groined arch, as compared with other types of arches in the above-mentioned respects, needs no demonstration, except as to the economy of material, which involves two considerations: First, the method of computing the amount of masonry in any system of vaulting; and second, the strength of that vaulting; which will now be discussed.

Assume, for purposes of illustration, a given square or rectangular filter area which is to be roofed over with a system of groined arches, as shown in Fig. 3, built of concrete, of which the amount is to be computed. It will be noticed that this roof is made up, first, of a series of groined arch units $DEFG$ (or $NOQR$, which is equivalent to it), of span equal to the distance in the clear between piers, and each supported by one pier, as $HIJK$; second, of a cloistered arch of span $2BC$ and $2AB$, made up of the four corners of the area, each corresponding to $ABCD$, joined together, and third, of half of a cylindrical arch extending around the entire area, springing from the retaining wall and ending at the crown of the vault between said retaining wall and the first line of piers, of length $2CW$ and $2AV$, respectively. The roof vaulting thus springs from and is supported by the side walls and piers, and forms crown lines parallel to and lying between the several rows of piers, and the piers and side walls, in both directions.

Let us take up, therefore, in turn, the computation of these units —the groined arch unit, the cloistered arch unit, and the cylindrical arch unit.

In considering the easiest method of computing the masonry in these units it has seemed to the writer—in view of the different materials of construction used, whether one or several kinds together, as brick or concrete, or both, and the diverse forms of extrados used,

often regardless of the materials of construction, according to which the extrados may conform to the soffit or be parallel to it, may be a horizontal plane surface passing through the crown, an inverted pyramid over the piers, or any other surface suiting the particular circumstances or the preference of the designer—that the simplest method was to compute the volume of masonry as that of a circumscribed prism from which has been deducted the volume of the air space between it (said circumscribed prism) and the enclosed masonry. This method has, therefore, been adopted, and in Table No. 1 will be found the values now to be deduced for the volume of the air space between the horizontal plane of the springing lines and the soffit of various unit-types of vaulting, by the aid of which, in the manner above described, the volume of masonry may be computed.

The Groined Arch Unit.

The groined arch unit of masonry may be defined, for the purposes of this discussion, as the masonry in a system of groined arch vaulting supported by one pier and contained within the four vertical planes passing through the central axes and crowns of the arches springing therefrom. This unit, shown in Fig. 4, is equivalent to the unit (Fig. 5) contained within four vertical planes passing through the centers of four adjacent piers, and becomes identical with it if the several parts of the latter be but rearranged.

The Segmental Groined Arch Unit.—Let us consider first the general case of the (circular) segmental groined arch unit—the semi-circular groined arch unit being but the particular case in which the rise of the arch is equal to half the span—and

Let a = half the span of the arch;
b = rise of the arch;
r = radius of the arch;
d = diameter of semi-circular arch;
m = length of supporting piers;
n = width of supporting pier;
w = width or length of pier if the latter be square;
t = thickness of masonry at crown.

Referring to Fig. 4, the volume of air space between the plane of the springing lines of the soffit and of the vaulting is equal to eight times the volume of the "$\frac{1}{8}$-unit" air space, so-called [being that seg-

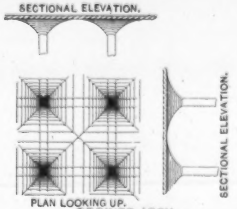


FIG. 1.

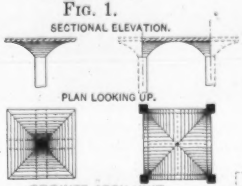


FIG. 4. FIG. 5.

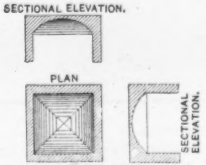


FIG. 2.

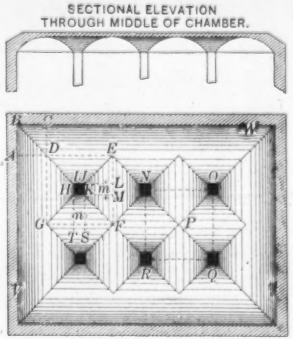


FIG. 3.

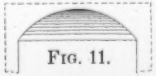


FIG. 11.

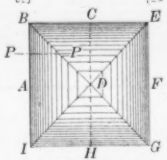


FIG. 12.

FIG. 13.

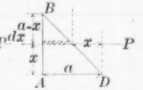


FIG. 14.

FIG. 6.

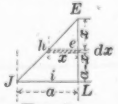
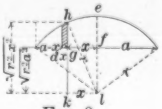


FIG. 7.



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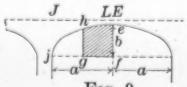


FIG. 9.

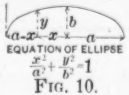


FIG. 10.

ment of a cylinder underlying the portion of the vaulting shown in plan by $J\,E\,L$ (Fig. 3), enclosed by a horizontal plane passing through the springing line, by the soffit of the arch and by three intersecting vertical planes passing through $J\,E$, $E\,L$ and $L\,J$], plus the volume of a segmental cylindrical arch of span $2\,a$, rise b, and length equal to $(m + n)$, the width plus the length of the pier.

The volume of the $\frac{1}{8}$-unit air space may be found in the following manner: Passing a vertical plane, perpendicular to $E\,L$, through any point distant x from E in the line $E\,L$, it will cut from the $\frac{1}{8}$-unit air space a quadrilateral $e\,f\,g\,h$, shown in Figs. 6 and 7, of which the line $e\,h$ is the segment of a circle, and the area of which is indicated by the expression:

$$\int_o^x \left(\sqrt{r^2 - x^2} - \sqrt{r^2 - a^2} \right) d\,x \quad \ldots\ldots (1)$$

$$= \frac{x}{2} \sqrt{r^2 - x^2} + \frac{r^2}{2} \sin^{-1} \frac{x}{r} - x \sqrt{r^2 - a^2} \quad \ldots\ldots (2)$$

as may be readily deduced from Figs. 6, 7 and 8.

The volume of air space underlying a $\frac{1}{8}$-unit of the (circular) segmental groined arch is therefore found by integrating the above expression (2) between the limits of a and 0, thus:

$$\int_o^a \left(\frac{x}{2} \sqrt{r^2 - x^2} + \frac{r^2}{2} \sin^{-1} \frac{x}{r} - x \sqrt{r^2 - a^2} \right) dx$$

$$= \frac{(r^2 - a^2)^{\frac{3}{2}}}{3} + \frac{a r^2}{2} \sin^{-1} \frac{a}{r} - \frac{r^3}{3} \quad \ldots\ldots (3)$$

$$= \frac{(r - b)^3}{3} - \frac{r^3}{3} + \frac{a r^2}{2} \sin^{-1} \frac{a}{r} \quad \ldots\ldots (4)$$

The volume of the segmental cylindrical arch, of span $2\,a$, rise b, and length $(m + n)$ is derived similarly to (1), and is indicated by the expression:

$$(m + n) \int_{-a}^{a} \left(\sqrt{r^2 - x^2} - \sqrt{r^2 - a^2} \right) d\,x$$

$$= (m + n)\,(r^2 \sin^{-1} \frac{a}{r} - a \sqrt{r^2 - a^2}) \quad \ldots\ldots (5)$$

or

$$= (m + n) \left[r^2 \sin^{-1} \frac{a}{r} - a\,(r - b) \right] \quad \ldots\ldots (6)$$

hence, the total volume of air space of the (circular) segmental groined arch unit between the plane of springing lines and soffit of arch is equal to $8 \times (4)$ plus (5) or (6).

$$= \frac{8}{3} (r - b)^3 - \frac{8}{3} r^3 + (4\,a + m + n)\, r^2 \sin^{-1} \frac{a}{r} - a\,(m + n)$$
$$(r - b) \quad \ldots\ldots (7)$$

If the pier is square and $m = n = w$, (7) becomes,

$$\frac{8}{3}(r-b)^3 - \frac{8r^3}{3} + 2r^2(2a+w)\sin^{-1}\frac{a}{r} - 2aw(r-b) \quad (8)$$

If the pier area becomes equal to 0, *i. e.*, a point, (7) becomes,

$$\frac{8}{3}(r-b)^3 - \frac{8r^3}{3} + 4ar^2\sin^{-1}\frac{a}{r} \quad (9)$$

With the aid of the above formulas the volume of the (circular) segmental groined arch unit may readily be found when the radius, span, rise and crown thickness of arch, dimensions of piers and form of extrados are known. Thus, using the same nomenclature as before (see page 42), if the spandrel of the arches be filled with masonry so that the extrados is a horizontal plane surface passing through the crown, at which the depth of masonry is equal to t, the volume of masonry in the (circular) segmental groined arch unit becomes

$$(2a+m)(2a+n)(b+t) - \text{formula (7)}$$

$$= (2a+m)(2a+n)(b+t) - \frac{8}{3}(r-b)^3 + \frac{8}{3}r^3 - (4a+m+n)$$

$$r^2\sin^{-1}\frac{a}{r} + a(r-b)(m+n) \quad (10)$$

or if the pier is square and $m = n = w$,

$$(2a+w)^2(b+t) - \frac{8}{3}(r-b)^3 + \frac{8}{3}r^3 - 2r^2(2a+w)\sin^{-1}$$

$$\frac{a}{r} + 2aw(r-b) \quad (11)$$

or if the pier area is 0, *i. e.*, a point,

$$4a^2(b+t) - \frac{8}{3}(r-b)^3 + \frac{8}{3}r^3 - 4ar^2\sin^{-1}\frac{a}{r} \quad (12)$$

If the extrados filling be depressed over the piers in the form of an inverted pyramid, the volume of masonry would be found by deducting from (10), (11) or (12) the volume of that inverted pyramid.

Or, finally, if the extrados also is segmental in form, *i. e.*, is parallel to the soffit of the vaulting, the volume of masonry in the (circular) segmental groined arch unit would be found by deducting the volume between the plane of the springing lines and the soffit of the vaulting, as determined by formulas (7), (8) or (9), from the volume between the plane of the springing lines and the extrados of the vaulting as determined by formulas (7), (8) or (9), substituting therein, for the values of the radius, span and rise of the soffit arch, those functions of the extrados arch, and, for the dimensions of piers, the new values determined from the section cut by the extrados from the pier at the plane of the springing lines.

Semi-Circular Groined Arch Unit.—Similar values for the semi-circular groined arch unit may be obtained directly by substituting in the formulas just deduced for the (circular) segmental groined arch the value of the semi-span a and rise b of the arch, their equivalent, the radius of the arch r or half the diameter, $\frac{d}{2}$, and are to be found in Table No. 1.

The Elliptical Groined Arch Unit.—In deducing general formulas for the computation of the masonry in the elliptical groined arch unit of vaulting, the same general methods of analysis can be used as were followed in the case of the (circular) segmental groined arch vaulting, the only difference being that the transverse curve of the arch varies according to the law of the ellipse, $\frac{x^2}{a^2} + \frac{y^2}{b^2} = 1$, instead of that of the circle $y = \sqrt{r^2 - x^2} - \sqrt{r^2 - a^2}$.

Referring to Figs. 9 and 7, the volume of air space of the $\frac{1}{8}$-unit elliptical groined arch between the plane of the springing lines and the soffit of the arch is

$$\frac{b}{a}\int_o^a x\sqrt{2\,a\,x - x^2}\;\;d\,x$$

$$= 0.4521\,a^2\,b \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (21)^*$$

which is analogous to the expression deduced for the semi-circular groined arch unit (see Table No. 1).

The volume of the elliptical cylinder, of span $2\,a$, rise b and length $m + n$, is

$$\frac{(m + n)\quad(\pi\,a\,b)}{2} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (22)$$

Hence the total volume of air space between the plane of the springing-lines and the soffit of the vaulting of an elliptical groined arch unit is, when the pier area is equal to $m \times n$,

$$3.6168\,a^2\,b + 1.5708\,a\,b\;(m + n) \ldots\ldots\ldots\ldots\ldots\ldots (23)$$

When the pier is square,

$$= 3.6168\,a^2\,b + \pi\,a\,b\,w \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (24)$$

When the pier is zero,

$$= 3.6168\,a^2\,b \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (25)$$

And the volume of masonry in the elliptical groined arch unit the extrados of which is a horizontal plane surface passing through the

* Formulas (13) to (20), inclusive, refer to the semi-circular groined arch unit, omitted here for lack of space, but found in condensed form in Table No. 1.

crown (at which point the masonry has a depth or thickness equal to t) is, when the pier area is $m \times n$,

$$(2a + m)\ (2a + n)\ (b + t) - 3.6168\, a^2 b - 1.5708\, a\, b\, (m + n) \ldots (26)$$

When the pier is square,

$$= (2a + w)^2\ (b + t) - 3.6168\, a^2 b - \pi\, a\, b\, w \ldots\ldots\ldots\ldots (27)$$

When the pier area is zero,

$$= 4\, a^2 t + 0.3832\, a^2 b \ldots\ldots\ldots\ldots\ldots\ldots (28)$$

Any difference in the form of the extrados or back of the vaulting may be treated in a manner precisely similar to that suggested for the segmental groined arch vaulting.

Elliptical Segmental Groined Arch Unit.—Formulas have not been deduced for this form of groined vaulting on account of the improbability of its being used; for if a segmental arch is required, the circular form would probably be used in preference to the elliptical. In any event, the deduction of formulas to cover this case would be similar to that already outlined for the circular segmental groined arch.

Parabolic Groined Arch Unit.—The volume of masonry involved in the parabolic groined arch unit—that is, a groined arch of which the right section through the crown is a parabola with vertical axis and vertex at the crown—may be computed in the same way as the circular and elliptical groined arch units; bearing in mind that the equation of the curve has to be modified to fit the parabola, $x^2 = \frac{a^2 y}{b}$.

The parabolic groined arch will be found most convenient for use as an invert for the floor of the reservoir or filter bed, on account of its similarity to the segmental groined arch, and the ease with which grade stakes can be set for it.

Referring to Figs. 9 and 7, the volume of air space of the $\frac{1}{8}$-unit parabolic groined arch between the plane of the springing lines and the soffit of the arch is:

$$= \frac{5\, a^2\, b}{12} \ldots\ldots\ldots\ldots\ldots\ldots (29)$$

The volume of the parabolic cylinder, of span $2a$, rise b, and length $m + n$, is

$$= \frac{4\, a\, b}{3}\ (m + n) \ldots\ldots\ldots\ldots\ldots\ldots (30)$$

Hence, the total volume of air space between the plane of the springing lines and the soffit of the vaulting of a parabolic groined arch unit is, when the pier area equals $m \times n$,

$$\frac{10\, a^2\, b}{3} + \frac{4\, a\, b}{3}\ (m + n) \ldots\ldots\ldots\ldots\ldots (31)$$

When the pier area is square,

$$= \frac{10\,a^2\,b}{3} + \frac{8\,a\,b\,w}{3} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (32)$$

When the pier area is zero,

$$= \frac{10\,a^2\,b}{3} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (33)$$

from which the volume of masonry may be computed as previously outlined.

Cloistered Arch Unit.

Let us now deduce formulas covering the same ground for the cloistered arch; the second step (see page 41) in the computation of the masonry of the hypothetical roof under discussion.

The cloistered arch unit differs from the groined arch unit, in that, whereas the groined arches spring from piers in points (or lines of length equal to the dimensions of the pier), and meet at the crown in intersecting lines of lengths equal to the span plus the width and length, respectively, of the pier, the cloistered arches spring from the side walls, or lintel arches, in lines circumscribing the unit, and intersect at the crown in a single point. Hence the same general method of computation and the same mathematical forms as were used for computing the masonry in groined vaulting may be used for cloistered vaulting, if only the proper limits for integration be substituted therein.

Segmental Cloistered Arch Unit.—Adopting the same nomenclature as before, we find in determining the volume of air space in the $\frac{1}{8}$-unit cloistered arch shown in plan by $A\ B\ D$, Figs. 12 and 14, that the vertical cutting plane $P\ P$, drawn perpendicular to $A\ B$ at a distance x from A, intersects therefrom a partial segment $g\,h\,j$, whose area,

if a = semi-span,

r = radius of curve of soffit of segmental cloistered arch,

$b = r - \sqrt{r^2 - a^2}$ = rise,

is equal to,

$$\int_x^a (\sqrt{r^2 - x^2} - \sqrt{r^2 - a^2})\,dx$$

$$= \frac{r^2}{2}\sin^{-1}\frac{a}{r} - \frac{r^2}{2}\sin^{-1}\frac{x}{r} + \frac{(2\,x - a)}{2}\sqrt{r^2 - a^2}$$

$$- \frac{x}{2}\sqrt{r^2 - x^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (34)$$

Hence the volume of air space between the plane of the springing lines and the soffit of the arch of the $\frac{1}{8}$-unit segmental cloistered arch is

$$\int_o^a [\text{Formula (34)}]\, dx$$

$$= \frac{(r^2 - a^2)^{\frac{3}{2}}}{6} - \frac{r^2}{2}\sqrt{r^2 - a^2} + \frac{r^3}{3} \quad \ldots\ldots\ldots\ldots (35)$$

$$= \frac{b^2}{6}(3r - b) \quad \ldots\ldots\ldots\ldots (36)$$

And the volume of air space between the plane of the springing lines and the soffit of the vaulting of a segmental groined arch unit, of span $2a$, and rise b, is

$$= \frac{4b^2}{3}(3r - b) \quad \ldots\ldots\ldots\ldots (37)$$

Semi-Circular Cloistered Arch Unit.—By making the rise (which is equal to the semi-span) equal to the radius of the arch in Formulas (34), (35), (36) and (37), for the segmental cloistered vaulting, we have corresponding formulas for the semi-circular cloistered arch (see Table No. 1)

Elliptical Cloistered Arch Unit.—Similarly, the volume of the air space under the $\frac{1}{8}$-unit elliptical cloistered arch is

$$\int_o^a \frac{b}{a}\left[\frac{(x-a)}{2}\sqrt{2ax - x^2} + \frac{a^2}{2}\,\text{vers}^{-1}\frac{x}{a}\right] dx \quad \ldots\ldots (38)$$

$$= \frac{a^2 b}{3} \quad \ldots\ldots\ldots\ldots (39)$$

And the volume of the air space under the elliptical cloistered arch unit is,

$$\frac{8a^2 b}{3} \quad \ldots\ldots\ldots\ldots (40)$$

Parabolic Cloistered Arch Unit.—The volume of the air space under the $\frac{1}{8}$-unit parbolic cloistered arch is,

$$= \int_o^a \left[\frac{2(a\ b)}{3} - b\ x + \frac{(b x^3)}{3a^2}\right] dx \quad \ldots\ldots\ldots\ldots (41)$$

$$= \frac{a^2 b}{4} \quad \ldots\ldots\ldots\ldots (42)$$

And the air space under the parabolic cloistered arch unit is

$$2a^2 b \quad \ldots\ldots\ldots\ldots (43)$$

By the aid of these formulas may be found, as previously described, the volume of masonry in any cloistered arch, by deducting from the computed volume of a circumscribed prism the volume of the air space underlying the soffit of the vaulting, together with the volume of the air space, if any, between the extrados and the horizontal upper base of the prism which passes through the crown. It

should be noted, moreover, that the dimensions (length and width) of the prism circumscribing the cloistered arch are not merely equal to the clear span in both directions, but are in each case greater than those spans by an amount equal to the combined thickness of the vaulting at the two opposite springing lines, and hence equal to the span plus twice the thickness of the abutments.

Cylindrical Arch Units.

Formulas for the volume of the air space between the plane of the springing lines and the soffit of the cylindrical arch units have already been deduced, but are gathered here for convenience:

For the segmental cylindrical arch,

$$= l\left[r^2 \sin^{-1}\frac{a}{r} - a\,(r - b)\right] \quad \ldots\ldots\ldots\ldots\ldots\ldots (44)$$

For the semi-circular cylindrical arch,

$$= \frac{\pi r^2 l}{2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (45)$$

For the semi-elliptical cylindrical arch,

$$= \frac{\pi a b l}{2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (46)$$

For the parabolic cylindrical arch,

$$= \frac{4}{3}\frac{a b l}{} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (47)$$

in all of which

l = length of cylindrical arch,
r = radius of arch,
a = semi-span of arch,
b = rise of arch.

Tables.

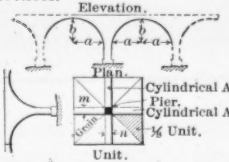
Table No. 1 contains, for convenience in computation, the formulas just deduced relating to the volume of the air space between the plane of the springing line and the soffit of the vaulting of the different types of vaulting discussed.

In Table No. 2 is given a comparison of the ratios of the volume of masonry in the groined, cloistered, cylindrical, and dome arch units, the pier area and width of abutment of which is equal to 0, and of which the extrados is a plane surface passing through the soffit crown to the volume of masonry in a circumscribed rectangular prism, passing through the same springing lines and soffit crown; which

TABLE No. 1. VOLUME OF AIR-SPACE BETWEEN SOFFIT OF ARCH & PLANE THROUGH SPRINGING LINES.

TYPE OF ARCH		SEGMENTAL	SEMI-CIRCULAR	ELLIPTICAL	PARABOLIC
Groined Arch.	Unit of Pier Area $m \times n$	$=\frac{8}{3}(r-b)^3-\frac{8r^3}{3}+(4a+m+n)r^2\sin^{-1}\frac{a}{r}-a(m+n)(r-b)$	$=\pi r^2(2r+\frac{m+n}{2})-\frac{8r^3}{3}$. $=\frac{\pi d^2}{4}(d+\frac{m+n}{2})-\frac{d^3}{3}$.	$=\frac{\pi ab}{2}(m+n)+8a^2b(0.4521)$ $=1.5708ab(m+n)+3.6168a^2b$.	$=\frac{10}{3}a^2b+\frac{4}{3}\frac{ab(m+n)}{3}$ $=3.333a^2b+1.333\frac{ab}{(m+n)}$
	Pier Area Square	$=\frac{8}{3}(r-b)^3-\frac{8r^3}{3}+2r^2(2a+w)\sin^{-1}\frac{a}{r}-2aw(r-b)$.	$=\frac{\pi d^2}{4}(d+w)-\frac{d^3}{3}$.	$=3.1416abw+3.6168a^2b$.	$=\frac{10}{3}\frac{a^2b}{3}+\frac{8}{3}\frac{abw}{3}$.
	Pier Area Zero	$=\frac{8}{3}(r-b)^3-\frac{8r^3}{3}+4ar^2\sin^{-1}\frac{a}{r}$.	$=8r^3(\frac{\pi}{4}-\frac{1}{3})=3.6168r^3$.	$=3.6168a^2b$	$=\frac{10}{3}\frac{a^2b}{3}=3.3333a^2b$.
	⅛-Unit.	$=\frac{(r^2-a^2)^{\frac{3}{2}}}{3}+\frac{ar^2}{2}\sin^{-1}\frac{a}{r}-\frac{r^3}{3}$ $=\frac{(r-b)^3}{3}-\frac{r^3}{3}+\frac{ar^2}{2}\sin^{-1}\frac{a}{r}$.	$=r^3(\frac{\pi}{4}-\frac{1}{3})$ $=0.4521r^3$.	$=a^2b(\frac{\pi}{4}-\frac{1}{3})$ $=0.4521a^2b$.	$=\frac{5}{12}\frac{a^2b}{12}$. $=0.4166a^2b$.
Cloistered Arch.	Unit.	$=\frac{4}{3}b^2(3r-b)$.	$=\frac{8r^3}{3}=2.6667r^3$.	$=\frac{8}{3}a^2b=2.6667a^2b$.	$=2a^2b$.
	⅛-Unit.	$=\frac{b^2}{6}(3r-b)$.	$=\frac{r^3}{3}=0.3333r^3$.	$=\frac{a^2b}{3}=0.3333a^2b$.	$=\frac{a^2b}{4}=0.25a^2b$.
Cylindrical Arch.		$=l\left[r^2\sin^{-1}\frac{a}{r}-a(r-b)\right]$	$=\frac{\pi a^2 l}{2}=\frac{\pi r^2 l}{2}$.	$=\frac{\pi abl}{2}$.	$=\frac{4}{3}\frac{abl}{3}=1.3333abl$.
Rectangular Prism		$=2abl=4a^2b$ when $l=2a$ $=(2a+m)(2a+n)(b+t)$	←	←	←

References:



a = semi-span of arch.
b = rise of arch.
r = radius of arch.
d = diameter of semi-circular arch.
m = length of supporting pier.
n = width of supporting pier.
w = length of square pier.
l = length of cylindrical arch.
t = thickness of concrete at crown.

Note: Air-space "unit" = space underlying portion of roof supported by one pier and contained within the four vertical planes passing through the central axes and crowns of the arches springing therefrom.

table, though misleading in certain respects, has, perhaps, a comparative value, and is to that extent suggestive.

TABLE No. 2.—Comparison of Masonry in Groined, Cloistered, Cylindrical, and Dome Arch Units; with Volume of Circumscribed Rectangular Prism passing through Springing Lines and Crown (of Soffit, not of Extrados) of Arch Unit.

Pier area and width of abutment = 0, and extrados a horizontal plane surface through soffit crown:

Description of "Unit."	Air space under masonry.	Volume of masonry. Cubic feet.	Ratio of volume of masonry to circumscribed rectangular prism.
Rectangular Prism	.....	$4a^2b$	100%
Elliptical Groined Arch Area of pier = 0	$3.6168a^2b$	$0.3832a^2b$	9.58%
Circular Groined Arch Area of pier = 0	$3.6168a^3$	$0.3832a^3$	$9.58\% \times \frac{a}{b}$
Parabolic Groined Arch Area of pier = 0	$3.3333a^2b$	$0.6666a^2b$	16.66⅔%
Elliptical Cloistered Arch	$2.6667a^2b$	$1.3333a^2b$	33⅓%
Circular Cloistered Arch	$2.6667a^3$	$1.3333a^3$	$33\frac{1}{3}\% \times \frac{a}{b}$
Elliptical Cylindrical Arch ..... $l = 2a$	$3.1416a^2b$	$0.8584a^2b$	21.46%
Circular Cylindrical Arch ..... $l = 2a$	$3.1416a^3$	$0.8584a^3$	$21.46\% \times \frac{a}{b}$
Hemispherical Dome of equivalent area .... $R = 1.1284a$	$2.0944R^3$	$1.0472R^3$	37.61%

References:
a = semi-span of arch.
b = rise of arch.
R = radius of arch.
L = length of cylindrical arch.

Table No. 3 contains a comparison of the amounts of masonry in the elliptical and semi-circular groined, cloistered and cylindrical arch units, with that in a circumscribed rectangular prism passing through the springing lines and the (extrados) crown of the vaulting, based upon the following, and, perhaps, more satisfactory assumptions:

Span of arch ..... $= 2a$;

Rise of elliptical arches ..... $= \frac{1}{4}$ of span $= \frac{a}{2}$;

Piers square;

Width of piers or thickness of abutments ..... $= \frac{1}{6}$ to $\frac{1}{24}$ of span;

Thickness of masonry at crown .. $= \frac{1}{12}$ of span;

Spandrels of masonry level with crown (that is, extrados crown).

The results are certainly interesting, and while the writer does not propose to discuss them here—as they speak for themselves—he would call attention to the fact that they are not strictly comparable for a large structure, for the reason that while the span, rise and crown thickness are the same in all cases, the depth of abutment or piers (and consequently the amount of masonry), varies, and for the further reason that no allowance has been made for the amount of

TABLE No. 3.—COMPARISON OF MASONRY IN GROINED, CLOISTERED AND CYLINDRICAL ARCH UNITS, WITH VOLUME OF CIRCUMSCRIBED RECTANGULAR PRISM, BASED UPON THE FOLLOWING ASSUMPTIONS: SPAN OF ARCH $= 2a$; RISE OF ELLIPTICAL ARCHES $= \frac{1}{2}$ OF SPAN; PIERS, SQUARE; WIDTH OF PIERS OR ABUTMENT $= \frac{1}{6}$ TO $\frac{1}{24}$ OF SPAN; THICKNESS OF CONCRETE AT CROWN $= \frac{1}{32}$ OF SPAN; SPANDRELS FILLED WITH MASONRY LEVEL WITH CROWN (OF EXTRADOS).

Description of arch.	Volume of masonry.	Volume of circumscribed prism.	Ratio %
Elliptical Groined Arch. Pier thickness $= \frac{1}{6}$ span	$0.729\,a^3$	$3.0625\,a^3$	23.8
Semi-circular Groined Arch. Pier thickness $= \frac{1}{6}$ span	$1.121\,a^3$	$5.785\,a^3$	21.0
Parabolic Groined Arch	$0.9514\,a^3$	$3.0625\,a^3$	31.6
Elliptical Cloistered Arch, with abutment thickness $= \frac{1}{12}$ span	$0.7292\,a^3$	$3.0625\,a^3$	23.8
Elliptical Cloistered Arch, with abutment thickness $= \frac{1}{6}$ span	$1.3333\,a^3$	$4.0000\,a^3$	33.3
Circular Cloistered Arch, with abutment thickness $= \frac{1}{12}$ span	$3.1180\,a^3$	$5.7847\,a^3$	53.9
Circular Cloistered Arch, with abutment thickness $= \frac{1}{6}$ span	$4.8889\,a^3$	$7.5556\,a^3$	64.7
Circular Cylindrical Arch, with abutment thickness $= \frac{1}{12}$ span	$2.1195\,a^3$	$5.7847\,a^3$	36.7
Circular Cylindrical Arch, with abutment thickness $= \frac{1}{6}$ span	$1.5844\,a^3$	$4.9878\,a^3$	31.8
Elliptical Cylindrical Arch, with abutment thickness $= \frac{1}{12}$ span	$1.2299\,a^3$	$3.0625\,a^3$	40.2
Elliptical Cylindrical Arch, with abutment thickness $= \frac{1}{24}$ span	$0.9889\,a^3$	$2.6406\,a^3$	35.6

The above comparisons are not absolute, as the dimensions of the circumscribing prism vary. Moreover, no allowance has been made in them for the masonry required for the lintel arches from which spring the cloistered and cylindrical arches.

masonry in the lintel arches required by the cloistered and cylindrical arches. This latter circumstance, however, is to the advantage of the groined arch in a consideration of its economy, as compared with other types of vaulting.

METHOD OF COMPUTING STRENGTH OF ARCH.

Turning now from the subject of the volume of masonry in the groined arch, let us briefly touch upon a method for studying or computing the stability of the groined arch.

Judging from the literature on the subject, but little study has been given to the stability of the groined arch by mathematicians and engineers in bygone years. Dr. Hermann Scheffler, in his "Theorie der Gewölbe Futtermauern und Eisernen Brücken," published in 1857, presented perhaps, at least so far as the writer has observed, the most complete analysis of the subject that has yet been published, but it is based, like his discussion of the cylindrical arch, on the assumption of the incompressibility of the building material. Rondelet, at about the same time, carried on a series of experiments with different types of arches upon small models, which experiments, though they seemed to bear out Scheffler's deductions, can hardly be accepted as more applicable to full-sized arches than are the early experiments of Hodgkinson and Fairbairn upon small models applicable to full-sized specimens of timber, iron and steel.

William Cain, M. Am. Soc. C. E., has also published a short article* on "Voussoir Arches Applied to Stone Bridges, Tunnels, Domes and Groined Arches" which will be found helpful, and which was written in continuation or amplification of his "Theory of Voussoir Arches."†

More recently, however, De Grande and Résal,‡ treating of "Groined Arches," have outlined a method which appears to the writer to be rational at least, if not so easy of application as an empirical formula, and which is herewith briefly abstracted and translated:

"* * * Let us consider, then, the conditions of stability which a work of this character presents.

"Let us suppose that we cut one of the two vaults by two vertical planes very close together, and normal to the generatrices $F\,G$ and $K\,L$, Fig. 255. The portion of the arch thus cut out is a cylindrical vault of infinitesimal length, and has for its right section, symmetrical with reference to the vertical plane through the crown, a fraction of the section of the complete cylinder $A\,B$ limited by the vertical planes $K\,F$ and $L\,G$. We can compute the thrust exerted by this arch on its abutments, and determine the point of intersection V of the line of resistance, in the plane of the section. Let P be the weight of this half incomplete arch and Q the thrust which it exerts.

"Let us consider the portion $L'\,G'\,K\,F$ of the arch which corresponds exactly to that previously considered on the adjacent vault.

* No. 42, Van Nostrand Science Series, 1879.

† No. 12, Van Nostrand Science Series.

‡ "Encyclopédie des Travaux Publics," Ponts en maçonnerie, par De Grande et J. Résal. Vol. 1, page 329

Considering the identity which exists between these two fractions of the arch from the point of view of the load, the length and the right section, the last will exert the same horizontal thrust Q, and its line of resistance will cut the vertical plane through $A\ C$ at the same point V.

"The lines of resistance of these two arches therefore meet in the same plane of the groin; the vertical components P of these pressures combine, the one with the other, and the horizontal components, that is to say, the thrusts which cut each other at right angles have for their resultant a horizontal force equal to $Q\sqrt{2}$, and contained in the vertical plane $A\ C$.

"Resuming now, the load supported by the portion of the groined arch $L\ G$, $F\ K$, $L'\ G'$ has for a result the development, in the plane of the groin $A\ C$, of a force of vertical component $2\ P$, and horizontal component $Q\sqrt{2}$, and applied at a point V, which we know how to find.

"Let us suppose that we divide the half of the vaulted arch $A\ B\ S\ D$ by a series of vertical planes, such as $F\ G$, $K\ L$, $F\ G'$, $K\ L'$, into a certain number of slices, and that we determine by the preceding method, in amount and direction, the resultant pressures transmitted by these slices in the plane of the groin $A\ S$.

"Fig. 256 represents the vertical section of the groined arch cut by the plane $A\ S$, and we have marked thereon the points of application $V_1\ V_2\ V_3$, * * * V_n of the resultants corresponding to the successive slices cut from the arch. It will suffice, to combine these different forces acting from the crown S toward the springing A, to obtain the curve of the line of resistance in the plane of the groin.

"The total weight ΣP transmitted to the upper section of the column will be equal to the weight of a quarter of the groined arch $A\ S\ B$, and the total thrust will be the sum of the partial thrusts calculated separately for the different slices.

"Let us cut the groin $A\ S$ by a plane $M\ N$, Fig. 255, which is perpendicular to it. Let W (Fig. 257) be the point of intersection in this plane of the line of resistance, and T the resultant of the reactions transmitted by the portion of the groined arch which lies on the groin between the crown S and the plane $M\ N$. This force T will distribute itself over a certain zone of the masonry, over one portion and another of the plane of symmetry $A\ S$, and as the total reaction of the arch will be definitely transmitted to the column A, it is natural to suppose that the zone of masonry affected by the force T will be sensibly limited by the vertical planes $R\ S$ and $T\ S$ limited by the crown and the opposite corners T and R of the column. This statement seems to us to be almost self-evident.

"Fig. 258 represents the section cut by the plane $M\ N$ from this prism of masonry which is called upon to balance by its molecular

action the force T. In order that the work may be stable it is necessary that the maximum pressure developed in this section should not exceed the practical limit; we can also calculate under all circumstances the value of this maximum pressure by the formulas for the resistance of materials.

"Three cases may present themselves: First, when the calculated pressure is admissible. The work will then have the desired stability without modification. Second, when the point W is too near the upper summit M, corresponding to the re-entrant angles of the profile. It is necessary then to reinforce the top of the section of masonry, and one can do this without difficulty (Fig. 258) by filling in the depression in the surface of the extrados of the groined arch in the plane $A\,S$. This reinforcement of the theoretical diedral angle by a regular surface, or one bringing the extradoses of the two vaults into accord, is very frequently practiced by builders. Third, when the point W is too near the inferior summit N corresponding to the groined edge of the work. This groin runs the danger, then, of failing and of crushing, in consequence of excessive compression. It is therefore necessary to reinforce it. This can be done by surrounding the groin with a prism of masonry forming a rib and intimately bonded to the mass of the groined arch (Fig. 259).

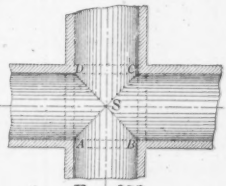


FIG. 253.

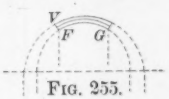


FIG. 255.

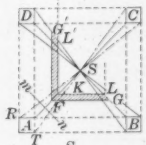


FIG. 256.

FIG. 258.

FIG. 257.

FIG. 259.

"This is a custom well known to architects, which is entirely justified, as we see, by the theory of arches. Sometimes this rib stops at its junction with the column, on the vertical ornament upon which it rests; or, again, it may continue from its junction with the column to the bottom of the latter, following the vertical edge A. Fig. 260 represents the plan of a work thus designed, and it is customary to place at the crown a salient stone on which the ribs of the groin S abut, and which forms a motive of decoration. * * * It happens sometimes that groined arches are formed by the intersection of two vaults of the same rise but different span. In this case the thrusts of their vaults being un-

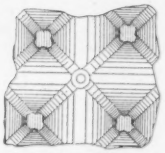


FIG. 260.

equal, the resulting pressures developed in their vertical plane of intersection will no longer necessarily be contained in this plane.

"The above-described method of analysis is perfectly applicable, however, though one recognizes in general that this disposition is less favorable to stability, and should be discarded when arches of large dimensions and heavy loads are concerned."

It is apparent, of course, that this method makes no allowance for the greater stability of the structure due to the monolithic character of the masonry when constructed of concrete; which stability is no doubt materially increased thereby.

Scheffler drew the following conclusions from his analysis of the stability of groined arches, as is to be found in his "Theorie der Gewölbe," page 183:

"It is a striking fact that, by comparison of these different values for E (the thickness of abutment), in all cases the thickness of abutment for a free standing pillar the value of which is computed according to (17), upon which two rib arches act at right angles simultaneously, is smaller than for a simple cylindrical arch, or for the unbalanced pressure resulting from a single arch rib. The reason for this is to be found in the fact that whereas in the first case, where all the forces unite along the diagonal, the moment of the abutment grows according to the $\sqrt{2} = 1.414$ times; under the same conditions, the moment of the injurious (harmful) horizontal forces, only increases at their junction; while the lever arm, therefore, also the moment of the vertical forces favorable to stability, increases in greater proportion, namely, as $\frac{3}{2}\sqrt{2} = 2.121$ times. * * *

"These results (which vary materially for greater coefficients of stability, as we have seen above) find a noteworthy confirmation in the practical investigations with models by Rondelet, which are described in his "L'Art de Bâtir," Book IX, 6th section, chapter 2 (page 327 of the German translation), and according to which, for the limit of equilibrium of equal span widths, the thickness of abutment of a dome arch, of a cloistered arch, of a cylindrical arch and of a groined arch, are about in the proportion of the numbers 1, 3, 4, 6."

Albert Hébrard in his book "L'Architecture" gives an empirical formula for the determination of the thickness of a supporting pier of a groined arch which reduces to the form: "$x = \sqrt{\frac{2\,F\,r\,L}{l}}$ when H is equal to or greater than the span" in which

x = thickness or depth of pier;

F = thrust at crown of arch between piers;

L = distance on centers of crown points, *i. e.*, span plus width of pier;

l = width of pier;

r = coefficient of stability, varying from 1 to 2, according to judgment.

But it will be noted that the thrust on the cylindrical arch has first to be determined, and the formula is at best only indirectly applicable.

Recent Examples in the United States.

But few examples of the use of the groined arch in its simplest form as a reservoir or filter bed covering are to be found, either in the United States or abroad. Their introduction for such purpose in this country was due to William Wheeler, M. Am. Soc. C. E., of Boston, who first made use of them in the construction of the covered sand filters designed by him for the Ashland (Wis.) Water Company in 1895 (see Plate II, Fig. 2). The groined arches there used were, as appears in Mr. Wheeler's paper on the Ashland filters, read before the New England Water-Works Association, March 10th, 1897:

"Elliptical arches of 15.75 ft. span and 3.50 ft. rise. The arch rings are about 5 ins. thick, consisting of two courses of bricks laid flatwise in Portland cement mortar. The spandrels of the arches and the spaces over the piers and adjacent walls are filled and covered with a backing of Portland cement concrete up to the general level of 4 ft. above the spring of the arches, but sloping down to a height of 2 ft. only, above the springing line at the rear of the outside wall."

The arches cover an effective area of about half an acre, in three compartments, and are protected by an earth embankment 2 ft. in depth covering them.

In 1896, groined arches were again used by the same engineer to cover another sand filter, designed by him for the Somersworth (N. H.) Water-Works (see Plate II, Fig. 1). These arches, which cover an effective area of half an acre, were also elliptical, of span 16 ft., rise 4 ft., built of one course of brick, laid on edge, backed with Portland cement concrete up to a horizontal plane; making the depth of masonry at the crown 6 ins., covered with from 2 to 2½ ft. of earth and gravel.

They were also used by Freeman C. Coffin, M. Am. Soc. C. E., in 1897, as a covering for a reservoir designed by him, and built for the Wellesley (Mass.) Water-Works upon Maugus Hill. These groined arches were also elliptical, constructed entirely of Portland cement concrete, of 12-ft. span, 2-ft. rise and with spandrels filled to a depth

giving 6 ins. at crown, covering a circular reservoir 80 ft. in diameter, and supporting an earth and gravel filling 2 ft. deep.

And, finally, Allen Hazen, Assoc. M. Am. Soc. C. E., has made use of them in the new and excellent sand filter plant recently designed by him, and now under construction, for the Albany (N. Y.) Water-Works.

These examples of the groined arch, in engineering structures of such a character, limited as they are in number, are all in the United States that have thus far (February, 1898), come to the knowledge of the writer, but it seems highly probable that with the great awakening in questions relating to bacteriological conditions of water supplies, the construction of covered filters and reservoirs will multiply. And for this reason the subject of groined arches as a covering for reservoirs and sand filters has seemed to the writer to be pertinent at this time.

DISCUSSION.

Mr. Le Conte. L. J. Le Conte, M. Am. Soc. C. E. (by letter).—This paper shows much careful study and research, while the tabulated results add very materially to its value. The writer will confine his remarks to the economic features of construction as applied to the covering of reservoirs and filters in general.

The masonry coverings considered by the author are extremely pleasing to the eye and are highly satisfactory in every respect except as to cost. There are many modern designs for masonry coverings, composed of concrete and iron tie-bars combined, which are equally durable, strong and efficient, take up less space inside and are cheaper and better in every way. Taking the author's limiting planes, for volumetric comparison, the modern designs with span of 25 ft. and thickness of only 2 ft. would require about 0.70 cu. ft. of Portland cement concrete per superficial foot of roof covering, and the expense would not exceed 50 cents per foot; if the spandrels were not filled in with masonry, the cost would not be more than 35 cents per superficial foot of roofing. Roughly speaking, 1 000 cu. ft. of good Portland cement concrete are enough for 1 875 sq. ft. of covering. Hence, the writer thinks that, in course of time, financial reasons, more than any others, will drive groined arches from the field of competition.

Mr. Hutton. William R. Hutton, M. Am. Soc. C. E. (by letter).—The author remarks that all the examples of the groined arch in engineering structures that have come to his knowledge are in the United States. These applications are not new in Europe, however. In 1863–65 Mr. Belgrand built the reservoirs of Menilmontant and Belleville in Paris, both of two stories. The lower story is covered with semi-circular groined arches of 16 ft. clear span on piers 4 ft. 8 ins. square. The arches are 14 ins. thick at the crown, and are leveled up with masonry. The upper reservoir is covered with groined arches of 18 to 20 ft. span, and 2 ft. 8 ins. rise; the arches being formed of two thicknesses of tile laid in cement, breaking joint. Their total thickness is less than 3 ins. Upon this, 18 ins. of earth and sod are laid.

Ten or more years later, the reservoir of Montsouris was built in the same way. The lower reservoir is covered with groined arches in rubble masonry 14 ins. thick at the crown; the haunches being filled with concrete or masonry. The upper reservoir is covered by a series of groined arches less than 3 ins. thick, on spans of 18 ft., with a rise of 2 ft. 8 ins. This thin arching is the "Guastavino" method, recently introduced into America by the architects. The writer has not investigated the origin of this system of arching as applied to reservoirs, which has been used by others than Belgrand, and in other places. The "Guastavino" method is much older. The Church of St. Eugene

(Paris), in 1854, was ceiled and roofed with Gothic ribs of iron, the arched surfaces between the ribs being of tiles laid flat, in cement. Mr. Hutton.

The upper reaches of the Canal St. Martin, in Paris, rest upon gypsum. The material in places has been dissolved out from under two of the upper locks and from one short level, by the filtration of water from the canal and from the Basin of La Villette. These have been repaired by means of piers sunk 30 ft. and 40 ft. to solid ground, and from them spring groined arches of brick or rubble to form the floor.

ALLEN HAZEN, Assoc. M. Am. Soc. C. E.—In looking over some plans and photographs of vaultings recently, the speaker was surprised to find how seldom the groined arch has been used, considering its marked advantages in economy and strength. Mr. Hazen.

In the Roman reservoirs of Constantinople* an arch resembling the groined arch was used freely, but it was not a true groined arch. It was made by springing narrow arches between the piers in each direction and dividing the area up into a series of squares which were covered by flat domes. This form of construction is often seen in architecture to-day. The ceiling of the reading room in the Public Library in Boston is a very handsome example of it.

An important modern application of this form of vaulting is at Warsaw, Russia, where filters and reservoirs of ten or twelve acres are covered in this way.

Many covered reservoirs and filters have been constructed with cylindrical arches resting upon narrow arches sprung between the piers in one direction only. This form of construction is common in England. The earliest Continental reservoirs and filters were also covered in this way. The older water-works vaultings at Berlin are of this class, and also the filters at St. Petersburg.

The more recent vaultings at Berlin, and elsewhere upon the Continent, are usually groined arches. The corners are often re-enforced with ribs, as Mr. Metcalf has suggested, and the speaker thinks there is a theoretical reason for this in the case of brick vaulting, although whether or not the additional strength is worth the trouble is another question. In the earliest covered reservoirs constructed in the United States, English precedent was followed, and cylindrical vaulting was used. The speaker thinks that the first large masonry-covered reservoir in America was at Newton, Mass. This reservoir was designed by the late Albert F. Noyes, M. Am. Soc. C. E., and Alphonse Fteley, Past-President, Am. Soc. C. E., acted as consulting engineer.

To William Wheeler, M. Am. Soc. C. E., belongs the credit of the introduction in America of the groined arch for water-works purposes, as has been stated by Mr. Metcalf. The speaker has had occasion to use it extensively in the new filters for the water-works of Albany, N. Y.,

* Forchheimer, "Wasserbehalter von Constantinopel," Wien, 1893.

Mr. Hazen. and has had occasion to make computations similar to those which Mr. Metcalf describes. The volumes of groined arches have been computed in the speaker's office by William B. Fuller, M. Am. Soc. C. E., and others, by methods similar to those which Mr. Metcalf has used. These computations have been checked by graphical methods. The graphical computation of the volumes is not difficult, and the results check perfectly those obtained by the formulas.

The speaker is sorry that the paper does not give more in reference to the strength of the vaulting; he thinks that all the discussion of the subject relates to brick arches. In the case of concrete arches he is inclined to think that the strains involved are somewhat different from those in brick, and that the computations relating to brick arches will have to be taken with some allowance when applied to concrete.

This, perhaps, will depend upon the way in which the concrete is placed. Concrete is necessarily placed in lots, leaving joints at certain places, and the tensile strength of these joints is very little, practically nothing. The speaker has used joints following the summits, that is, dividing the vaulting up into a series of squares, each having a pier as a center. Under these conditions, and with the dimensions used at Albany, it is a question whether there is any arch action whatever in the vaulting. Changes in temperature expand and contract the concrete. With brick arches this expansion and contraction would probably result in a slight rise and fall of the crowns, corresponding to the differences in length of the material in the arches. With the concrete, the action is apparently different. The blocks are so rigid that there is no appreciable rise and fall of the crowns. With contraction, the joints will open slightly in some cases, making it perfectly obvious that there is no pressure across them, and that the concrete blocks act as cantilevers; while with expansion, which occurs with concrete put in in cold weather and afterward warmed by summer heat, the whole mass of concrete has to push out at the sides. In some cases the cylindrical arch next to the outside wall may act as an expansion joint for the groined vaulting in the interior.

The speaker has mentioned the cantilever principle simply as a proposition. He is not prepared to say that he thinks all vaulting acts in that way. He is not fully informed as to the method of construction used in the Wellesley vaulting, but his impression is that the concrete was not placed in the manner used at Albany, and that the joints were made at other places; and if that was the case it could not be expected to act in the same manner. On the Albany work the concrete was mixed by machinery, and the amount of concrete required to fill one of the squares was placed in a very few minutes, and made a solid block, without a joint of any kind.

Regarding the practical strength of the concrete vaulting, there

can be no question. At first the speaker made certain rules in regard to teams driving over it, etc., but found it very hard to keep the contractors in order in that respect. After a little, teams were allowed to go over it freely, and also trucks, and later, rollers weighing three tons were taken over it repeatedly, and sometimes left on it, and all without any injurious results. Mr. Hazen.

The cost of the Albany vaulting per square foot of area was as follows:

Concrete 0.78 cu. ft. at $6.30 per cubic yard, including centers	18.20	cents.
Piers	5.44	"
Earth filling and seeding	1.42	"
Manholes, entrances, fasteners, etc.	2.65	"
Total	27.71	"

The total cost, including piers, was less than 28 cents per square foot. Various forms of steel and concrete construction were considered, before deciding upon the concrete vaulting, but the simple masonry was selected.

WILLIAM B. FULLER, M. Am. Soc. C. E. (by letter).—It is well known that there are many advantages to be derived from a covered reservoir or filter-bed as compared with one open to the wind, rain and light, and the fact that a covering of such a nature is seldom used would indicate a false impression, even on the part of many engineers, that it is necessarily very expensive and should be classed among the luxuries, except in cases where it is essential to prevent the formation of ice or algæ. Mr. Fuller.

This is due perhaps to the fact that engineers, in designing masonry coverings, have been hampered by precedent. They look back to structures, some of which, as the author points out, date from the time of the Roman supremacy, and proceed to construct vaulting as laid down on lines developed 2 000 or more years ago. This is particularly true of recent vaulting in Europe, outside of France, and the engineer, even if he should desire to branch out on a more rational design, is confronted with these examples of vaulting, used merely for the protection of water from the action of the elements, but actually strong enough to carry railroad freight traffic.

The modern development of Portland cement concrete furnishes the engineer of to-day with a structural material easy to handle, particularly adapted to the construction of vaulted masonry, and differing widely in some of its properties from materials heretofore commonly used in such construction; and the writer wishes to emphasize a few points concerning the stresses which result in monolithic work of this nature.

Mr. Fuller. The author translates and endorses a theory on the stability of groined arches dependent entirely on the non-existence of other forces than compression, and says :

"It is apparent, of course, that this method makes no allowance for the greater stability of the structure due to the monolithic character of the masonry when constructed of concrete ; which stability is no doubt materially increased thereby."

The writer has repeatedly noticed this tendency with engineers in designing structures of concrete ; they are made similar to structures in brick or stone, and the added stability due to other properties than those possessed by such materials is dismissed with the thought that the structure is all the safer. We would not think of substituting steel for cast iron and taking into account only the properties of the latter.

Monolithic concrete is an independent building material, and surely, in such a position as in groined arch masonry, is entitled to separate consideration ; the stresses in such a structure must be very complex, but possibly their nature is somewhat as follows :

Consider a groined arch unit as shown by the author in Fig. 4, and cover its upper surface with a uniformly distributed load; the result must be a tendency to lower the outside edges, and, in a single unit, such as we are now considering, it can be seen that this tendency must be resisted by the tensile, cross-breaking or shear stresses in the mass of the unit itself. This action will be understood more readily if we think of the unit as made up of a stack of square, thin plates of varying areas piled symmetrically on top of the pier, as in Fig. 15. If we go a step further, and remove the center of these plates, we have the result indicated in Fig. 16, in which most of the cross-breaking stresses are replaced by tension in the rings, and we have a case somewhat analogous to that of a dome.

FIG. 15.

The placing of these units together to form a groined arch vaulting also brings into play compressive stresses, but as there must be a perceptible lowering of the keystone before the arch action takes effect, it is seen that, if the groined arch unit is capable of resisting the tensile stresses, the arch action may not be brought into play at all and is certainly only a secondary consideration. In such a case the stability of the structure is added to in just such proportion as these tensile and other stresses are brought into play.

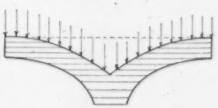


FIG. 16.

These illustrations are enough to induce a disbelief in the applicability of the author's theory to monolithic concrete vaulting, and the writer believes such a theory is equally inapplicable to brick or stone construction because the strain represented above as being

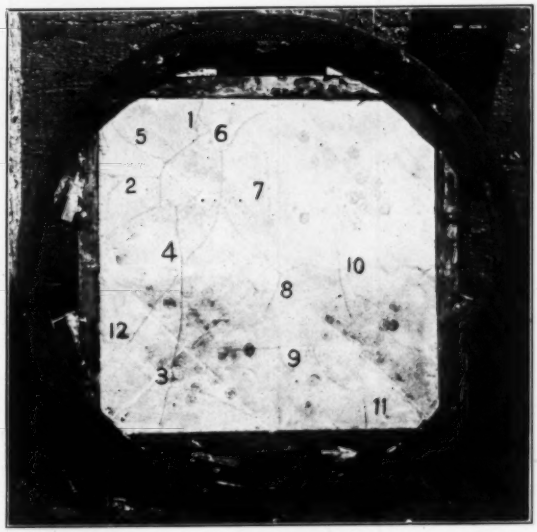


FIG. 1.—TOP VIEW, MODEL OF GROINED VAULTING.

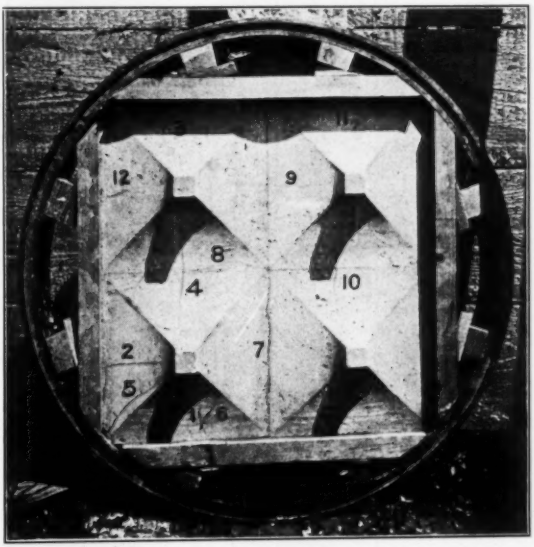


FIG. 2.—BOTTOM VIEW, MODEL OF GROINED VAULTING.

resisted by the mass of the unit section, in this other case would be resisted by the outside abutments of the structure. Mr. Fuller.

It is to be regretted that in demolishing a theory on which formulas are based, other formulas cannot be set up in their place, but the writer believes that the case of the groined arch is too complicated for the development of theoretical formulas which can be even approximately correct, and suggests that practical experimenting and the development of empirical formulas is the only solution of such questions.

The writer has experimented on models of groined vaulting constructed of neat Portland cement on a scale of $\frac{1}{12}$ of an actual structure and presents in Plate III, two photographs of one model showing the method of failure, which was nearly identical in all the models. In Plate III, Fig. 1 shows the top and Fig. 2 the underside of the model.

The cracks are numbered in the order in which they appeared as the loading was applied, and it will be seen that the first six were tension cracks, and, furthermore, that all units indicated tension cracks before the compression cracks appeared on that unit.

The surprising thing about these models was, that they sustained far heavier loading before showing failure than would be supposed possible, and thus indicated that the accepted sections for such vaulting can be reduced materially, but, of course, experiments should be tried on full-sized sections before any general deductions can be made.

It can be seen also that if iron in tension is used in the proper place in a groined arch, the strength of the arch will be increased materially, and, conversely, that its section and cost for a given strength can be reduced materially.

A little time and money devoted to experimenting along the lines indicated would put us in possession of a light, durable and comparatively inexpensive covering for reservoirs and filter beds.

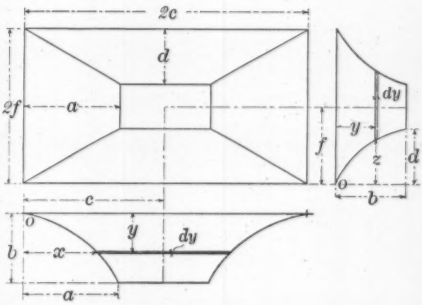


FIG. 17.

Concerning the calculation of quantities of the masonry vaulting the writer evolved formulas for the different cases some time ago, which gave the same results as those of the author, but, as his treatment of the subject differs from that of the author, a solution of one general case is appended to illustrate the difference in methods.

Mr. Fuller. Take the case of the parabolic groined arch unit as shown in Fig. 17, and the following notation :

	Longitudinal Span.	Cross Span.
One-half distance center to center of piers...	c	f
One-half clear span.....	a	d
Rise of span.....	b	b

With origin at O the equations of the generating parabolas are

$$x = \sqrt{\frac{a^2}{b}}\, y,\ z = \sqrt{\frac{d^2}{b}}\, y$$

and

$$c - x = c - \sqrt{\frac{a^2\, y}{b}},\ f - z = f - \sqrt{\frac{d^2\, y}{b}}$$

The area of any dy plane above is $4\,(c - x)\,(f - z)$. The volume of any portion of the solid is

$$V = \int_{y_1}^{y_2} 4\left(c - \sqrt{\frac{a^2\, y}{b}}\right)\left(f - \sqrt{\frac{d^2\, y}{b}}\right) dy$$

$$= 4\left[c f y - \frac{2\, y^{\frac{3}{2}}}{3\sqrt{b}}\,(a f + c\, d) + \frac{a\, d\, y^2}{2\, b}\right]\Bigg|_{y_1}^{y_2}$$

with limits of b and o.

$$V = 4\, b\left[c f - \frac{2}{3}\,(a f + c\, d) + \frac{a\, d}{2}\right] \ldots\ldots\ldots\ldots\ (1)$$

The volume of the air space remaining, between the plane of springing lines and soffit of arch, is

$$v = 4\, b\, c f - V = \frac{2\, b}{3}\,(4\, a f + 4\, c\, d - 3\, a\, d) \ldots\ldots\ldots\ (2)$$

The value of v in Formula (2) is the same as that found by the author after the necessary changes have been made in the notation.

Mr. Metcalf. Leonard Metcalf, Assoc. M. Am. Soc. C. E. (by letter).—Mr. Le Conte's statement that "there are many modern designs for masonry coverings, composed of concrete and iron tie-bars combined, which are equally durable, strong and efficient, take up less space inside and are cheaper and better in every way" is broad and sweeping. Expensive the groined arch masonry roofs doubtless are, though by no means always so when considered from a comparative point of view and the conditions they are called upon to meet. With the present prevailing high prices of iron and steel, for instance, they will be found more economical in many cases than steel and concrete construction. Freedom from the danger of corrosion, and cheapness of maintenance they certainly possess, together with strength and stability. Further study and experiment will doubtless determine the limits to which the dimensions of the arch may be safely reduced under different loads, with resulting economy of material.

Mr. Metcalf.

As regards the actual expense, Mr. Hazen has stated that the cost of the Albany filter plant roof amounted to only $0.182 per square foot for the concrete masonry in place, and less than $0.28 per square foot, including the cost of piers, earth filling and seeding, manholes, entrances, fasteners, etc. In a small structure, where the centering cannot be used a second time, the cost is relatively greater. Thus, the centering alone of the Wellesley reservoir is stated by its engineer, Freeman C. Coffin, M. Am. Soc. C. E., to have cost $0.225 per square foot. The roof of the Concord, Mass., sewage storage well, of 57 ft. internal diameter and containing about 100 cu. yds. of masonry, designed by the writer, cost for

Centering	$0.18	per square	foot.
Concrete, materials	0.15	"	"
Labor and supervision	0.05	"	"
Total	0.38	"	"

Mr. Hutton has, unintentionally no doubt, misquoted the writer in saying "all the examples of the groined arch in engineering structures that have come to his knowledge are in the United States." What the writer said, was: "these examples, * * * limited as they are in number, are all in the United States that have thus far come" to his knowledge. The limits of the paper forbade reference to the many examples, to be found at home and abroad, of the use of the groined arch in ecclesiastical structures, and the comparatively few in engineering structures, to several of which Mr. Hutton has interestingly referred. One or two structures, in addition to those described by Mr. Hutton, which have come to the writer's notice in the course of his reading, are perhaps worthy of note. The groined roof-arches covering the filter-beds of the Zurich, Switzerland, water-works,* which are segmental, 14 ft. 9 ins. span, 4 ft. 1 in. rise, and 8 ins. thickness of concrete at crown; and those of the Berlin water-works reservoir at Charlottenburg, referred to in William Morris's paper on "Covered Service Reservoirs."†

In the United States at least two more groined arch reservoirs have been built in the past year, both for the storage of sewage—one by the Metropolitan Water Board of Boston, at Clinton, Mass., the other by the writer at Concord, Mass.

The investigations relating to groined arches by Mr. Hazen and Mr. Fuller, made in the course of the design and construction of the Albany filter plant, and the subsequent development of certain contraction cracks in that structure under changing temperature conditions, are most instructive and worthy of study. The writer is

* *Engineering News*, July 12th, 1894; and *Minutes of Proceedings*, Institution of Civil Engineers, Vol. cxi, 1892-93.

† *Minutes of Proceedings*, Institution of Civil Engineers, Vol. lxxiii, pages 1-60.

Mr. Metcalf. inclined to agree with Mr. Fuller that tension in the masonry over each pier, acting upon a principle "somewhat analogous to that of the dome," may be a factor in the strength of the arch; and, as Mr. Hazen has suggested, that the cantilever principle, as well as that of the beam and that of the arch, is called into the play. Just where one action begins and the other leaves off cannot be determined or demonstrated, but it seems very probable that tensile stresses are first called into play in the structure, and are followed by compressive stresses under which the arch finally fails, as was indicated so clearly by Mr. Fuller's experiments upon small models. This indicates that the proper place to introduce steel rods into the roof to strengthen the masonry is, not over the piers, but along the crown lines across which tension cracks first appeared, in the models tested, before the compression forces were called into play.

Mr. Fuller's method of computing the volume of masonry in any given arch is interesting. The work involved appears to be substantially the same as in the method pursued by the writer.

Vol. XLIII. JUNE, 1900.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

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No. 868.

TEST OF A MECHANICAL FILTER.

By EDMUND B. WESTON, M. Am. Soc. C. E.

PRESENTED NOVEMBER 1ST, 1899.

WITH DISCUSSION.

The object of this paper is to describe briefly the results of a three-months' test of a mechanical filter, which has recently been installed for the East Providence Water Company, at East Providence, R. I.

The New York Filter Manufacturing Company, of New York City, furnished the filter, which is of a type known as the Jewell Gravity Filter.

The entire work, including the filter house and pure-water well, shown on Plate IV, Figs. 1 and 2, was designed and built under the direction of the writer, who acted as Consulting Engineer for the New York Filter Manufacturing Company.

The average daily quantity of water furnished to its consumers by the East Providence Water Company, at the present time, is about 200 000 galls. The available daily capacity of the filter is 500 000 galls., and the rate of filtration 125 000 000 galls. per acre per 24 hours. The filter house, pure-water well and pipes are arranged for a future addition of three more filters of the same design and capacity, when demanded by the requirements of the service.

The filter was run throughout the test under ordinary working conditions, and the filtered water was pumped directly into the mains and supplied to the consumers. The filter was in charge, under the writer's direction, of the regular pumping engineer of the East Providence Water Company. The writer gave directions from time to time, but did not, on the average, visit the filter plant more than once a week during the test.

The chemical analyses were made by Professor John Howard Appleton, of Brown University. The bacteriological work was done by Dr. Gardner T. Swarts, Secretary of the State Board of Health of Rhode Island, who also determined the color and alkalinity of the samples.

The samples of water for analysis were collected by the pumping engineer at about 8 o'clock in the morning, the filter-bed generally being washed about two hours earlier.

In a report, dated March 12th, 1894, describing experiments with experimental filters, made under the writer's direction, in Providence, from February, 1893, to January, 1894, inclusive, the writer's conclusions are given to the effect that water can be as satisfactorily purified by first-class mechanical filtration as by slow sand filtration.

Since this report was published, elaborate investigations, made with mechanical filters, at Louisville, Pittsburg and Cincinnati, with waters widely different from the water supply of Providence, have practically substantiated the writer's conclusions, as given in the Providence report. It would seem, therefore, that after taking into consideration the additional experimental results given in this paper, which were obtained with a filter absolutely in practical service, as though there could no longer be any reasonable doubt, if such may have existed, in regard to the practicability and efficiency of mechanical filtration, and that henceforth, in the broad field of water purification, mechanical filtration can be looked upon as being equally as desirable as slow sand filtration.

The chemical used during the test was sulphate of alumina, which was added to the raw water in the form of a coagulant solution, prepared by dissolving one part of sulphate of alumina in about 20 parts of filtered water. The solution was always thoroughly mixed before being used.

FIG. 1.—FILTER HOUSE, EAST PROVIDENCE WATER COMPANY.

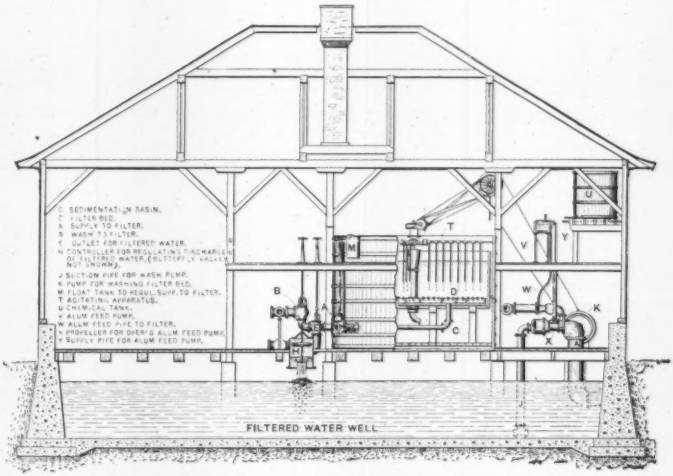

FIG. 2.—SECTIONAL ELEVATION THROUGH FILTER HOUSE.

The theory of mechanical filtration, when sulphate of alumina has been added to the filtered water, may be described briefly as follows: The alumina causes an artificial precipitation; a portion of the alumina is decomposed, forming sulphates of other bases and a flocculent precipitate of aluminic hydrate. A portion of it also combines directly with the organic matter in the water, coagulating the same and thus helping to increase the precipitation. The degree of color of the water is also largely reduced by the uniting of the precipitated aluminic hydrate with the coloring matter in the water.

The filter was first put in service on February 25th, 1899, and it has been in regular operation since that time. The test was commenced on March 13th, which was as soon as a known grade of sulphate of alumina could be procured. Previous to March 13th, sulphate of an inferior grade was used, and was added to the raw water at rates of from $\frac{1}{2}$ to $\frac{3}{4}$ grain per gallon. This was the only quality of sulphate of alumina which could be purchased in Providence at the time, and the percentage of $Al_2 O_3$ which it contained was not known.

The sulphate of alumina used during the test and special experiments, contained about 22% of $Al_2 O_3$, with the exception that during one special experiment an inferior and cheaper grade containing about 17.53% of $Al_2 O_3$ was used.

The sulphate of alumina was added to the raw water at the rate of 1 grain per gallon, with the exception that in three special experiments $\frac{3}{4}$, $\frac{n}{10}$ and $\frac{3}{4}$ grain per gallon were used.

Tables Nos. 1, 2, 3 and 4 give all the results obtained during the test, with the exception of those of the special experiments, which are given in Table No. 5.

As can be seen by Tables Nos. 2, 3 and 4, the investigations relative to the chemical constituents and the color and alkalinity of the samples were concluded on May 31st.

It is the intention to continue the bacteriological analyses, at least until June 30th, 1899, and Table No. 1 gives all the results which have been obtained in time to be made use of in this paper, with the exception of those of the special experiments.

Fig. 1, Plate IV, is a view of the filter house, which is located adjacent to the pumping station of the East Providence Water Company, at "Hunt's Mills." On the right of the plate can be seen a small portion of one end of the pumping station. The power required

for driving the agitator and wash pump is furnished by a turbine wheel located in the basement of the pumping station, and is transmitted to the filter house as shown on the plate.

Fig. 2, Plate IV, is a sectional elevation through the filter, filter house and pure-water well. The filter and auxiliaries comprise a sedimentation basin, a crushed quartz filter-bed having an area of about 176 sq. ft., a pump for adding the coagulant to the raw water, an automatic controller connected to the main discharge pipe of the filter for maintaining a constant rate of filtration, a pump and appliances for washing and agitating the filter-bed and for washing out the sedimentation basin, and screens connected to collecting pipes at the bottom of the filter-bed. The raw water, to which the sulphate of alumina has been added, enters the sedimentation basin through the valve on the supply pipe at *A*, and is deflected by a curved casting in such a manner that it is caused to circulate slowly around the basin. The water rises from the sedimentation basin through the central pipe shown in Fig. 2, Plate IV, to the required height above the filter-bed. The water passes downward and outward through the filter-bed, screens and collecting pipes to the main discharge pipe and controller, during the process of filtration, and inward and upward through the collecting pipes, screens and filter-bed, when the filter-bed is being washed. The screens prevent the quartz or any foreign substances from entering the collecting pipes and passing off with the filtered water.

The depth of the crushed quartz filter-bed is about 3.67 ft. The sedimentation basin has a capacity equal to a flow of about 17 minutes when the filter is being operated at the normal rate of 125 000 000 galls. per acre per 24 hours.

The coagulant pump is made of vulcanized rubber, and consists of six hollow arms radiating from a chambered hub, and bent in the direction of rotation. It is actuated by a propeller, situated in the main supply pipe, by the aid of an upright shaft and bevel gears. The pump gives excellent satisfaction, and its displacement is remarkably accurate.

The automatic controller is of a new and original design, and is a decided success. A careful test of the controller was made when it was first put in service, the flow of water being gauged accurately, and it was found, during a five-hour run of the filter, that the vari-

ation in the rate of filtration between the commencement and the end of the run was less than one-half of 1 per cent.

The manner in which the filter is operated is as follows: At the end of a run, or immediately before starting the filter, the filter-bed is washed thoroughly by forcing up through the screens and filter-bed a reverse flow of filtered water under pressure, the mechanical rake or agitator being operated at the same time, which adds materially to the efficient cleansing of the filter-bed. The water is forced up through the bed and the agitator is kept in motion until the water flowing from the overflow drain pipe is as clear as it was before being used for washing the filter. The necessary valves are then operated, and the unfiltered water, to which the sulphate of alumina is being added, is turned on the filter. The sedimentation basin is washed out by allowing the wash water at the top of the filter-bed to pass down into the basin through the central pipe shown in Fig. 2, Plate IV, and thence out through a waste pipe at the bottom of the basin. The agitator shaft runs through the central pipe and carries at its lower end a curved nozzle. As the agitator shaft is revolved, the nozzle is given a circular motion, and the rapid current of water passing through it is thrown to all parts of the sedimentation basin and stirs up and flushes out the accumulated sediment. The sedimentation basin is also provided with a manhole.

The averages of the results of Tables Nos. 1, 2 and 3 show that by the process of filtration there was:

99.20% less bacteria in the filtered water than in the raw water;
6 % less total solids in the filtered water;
1 % less chlorine in the filtered water;
61 % less ferric oxide in the filtered water;
38 % less aluminic oxide in the filtered water;
29 % less free ammonia in the filtered water;
63 % less albuminoid ammonia in the filtered water;
83 % less color in the filtered water;
20 % increase of hardness in the filtered water.

Table No. 4 shows that in every instance the filtered water was more or less alkaline, and, consequently, that the raw water was sufficiently alkaline to more than decompose the 1 grain per gallon of sulphate of alumina added to it.

Special attention is also called to the fact that Table No. 1 shows that the average number of bacteria per cubic centimeter found in the samples of filtered water is less than 5, and that the above summary shows that the filtered water contained 38% less alumina than did the raw water before the sulphate of alumina was added to it.

TABLE No. 1.—Bacteriological Analyses of Samples, by Dr. Gardner T. Swarts.

Results obtained during the time that 1 grain of sulphate of alumina per gallon was used.

Rate of filtration, 125 000 000 galls. per acre per 24 hours.

Date.	Bacteria per Cubic Centimeter.		Percentage of Reduction.	Date.	Bacteria per Cubic Centimeter.		Percentage of Reduction.
	Raw Water.	Filtered Water.			Raw Water.	Filtered Water.	
Mar. 13.....	768	4	99.49	Apr. 27.....	422	9	97.87
" 14.....	595	5.5	99.08	" 28.....	280	2.5	99.11
" 15.....	Steri lized	fil ter-bed.		" 29.....	370	6	98.38
" 16.....	1 299	9	99.31	May 8.....	266	9	96.62
" 17.....	1 257	7	99.45	" 9.....	976	3	99.69
" 18.....	683	4	99.41	" 10.....	708	13.5	98.09
" 31.....	782	1.5	99.82	" 11.....	150	5	96.66
Apr. 1.....	499	7	98.60	" 12.....	466	3.5	99.25
" 3.....	636	1.5	99.76	" 13.....	305	4	98.69
" 4.....	628	2	99.68	" 15.....	225	1	99.56
" 5.....	545	4	99.27	" 16.....	238	0.5	99.79
" 6.....	855	3	99.65	" 17.....	306	0.5	99.83
" 7.....	1 910	19	99.01	" 18.....	473	0	100.00
" 8.....	1 009	6.5	99.36	" 19.....	210	0.5	99.76
" 10.....	1 175	6.5	99.45	" 20.....	228	1	99.56
" 11.....	943	9.3	99.01	" 22.....	238	0.5	99.79
" 12.....	1 443	9	99.38	" 23.....	279	1	99.64
" 13.....	336	4.3	98.73	" 24.....	228	1	99.56
" 14.....	Lost.	4	.....	" 25.....	275	0	100.00
" 15.....	798	1.6	99.84	" 26.....	270	0.5	99.81
" 17.....	765	7.5	99.02	" 27.....	185	1	99.46
" 18.....	578	1.5	99.74	" 29.....	454	4.5	99.01
" 19.....	865	11	98.73	" 30.....	334	11.5	96.56
" 20.....	546	3	99.45	" 31.....	458	10	97.82
" 21.....	699	2	99.71	June 8.....	331	4	98.79
" 22.....	499	3	99.40	" 9.....	494	5.6	98.86
" 24.....	203	3	98.98	" 10.....	341	6.3	98.15
" 25.....	697	0.5	99.93	" 12.....	354	0.3	99.92
" 26.....	724	11	98.48	" 13.....	243	2	99.18
				Averages.....	570	4.5	99.20

Special experiments were made from March 20th to 30th, from May 1st to 6th and from June 1st to 7th.

The bacteria in the different samples were cultivated during periods of from five to six days.

The results of the special experiments mentioned are shown in Table No. 5.

As may be noticed, the percentage less of bacteria from May 1st to 6th, when ¾ of a grain of sulphate of alumina per gallon was used, is considerably less than it was from March 20th to 25th, when the same quantity of sulphate of alumina was being used. The difference may be accounted for partially by the fact that the average number of bacteria

TABLE No. 2.—Chemical Analyses of Samples, by Professor John Howard Appleton.

Results obtained during the time that 1 grain of sulphate of alumina per gallon was used.

Rate of filtration, 125 000 000 galls. per acre per 24 hours.

The numbers express parts (by weight) in one million parts of water (by weight).

Date.	Total solids.	Total hardness.	Chlorine.	Ferric oxide.	Aluminic oxide.	N as free ammonia.	N as albuminoid ammonia.	N as nitrates.	N as nitrites.
Raw Water.									
Apr. 6.....	39.0	16.0	4.8	0.58	0.47	0.04	0.22	0.60	Trace.
" 13.....	39.3	14.0	7.0	0.60	0.80	0.10	0.26	0.90	"
" 20.....	39.9	17.0	6.4	0.61	1.05	0.05	0.26	0.70	"
" 27.....	43.7	18.0	6.2	1.00	0.75	0.02	0.22	0.70	"
May 11.....	53.1	21.0	6.4	0.91	1.84	0.03	0.38	0.60	0
" 18.....	54.9	21.0	6.1	1.01	0.34	0.03	0.34	0.60	Trace.
" 25.....	49.6	20.0	6.4	1.09	0.76	0.04	0.32	0.60	"
Averages.....	45.6	18.1	6.2	0.83	0.86	0.04	0.29	0.67	.....
Filtered Water.									
Apr. 6.....	38.9	22.0	4.8	0.61	0.44	0.04	0.10	0.60	Trace.
" 13.....	40.4	19.0	6.0	0.45	0.55	0.05	0.11	0.70	"
" 20.....	37.4	22.0	6.4	0.28	1.02	0.05	0.10	0.60	"
" 27.....	39.6	19.0	6.2	0.19	0.26	0.01	0.07	0.60	"
May 11.....	47.2	23.0	6.3	0.20	0.55	0.02	0.14	0.40	0
" 18.....	49.8	23.0	5.8	0.13	0.47	0.03	0.12	0.50	Trace.
" 25.....	46.5	24.0	7.2	0.40	0.45	0.02	0.13	0.60	"
Averages.....	42.83	21.7	6.1	0.32	0.53	0.03	0.11	0.57	.....
Average Percentage, More or Less, in the Filtered Water.									
	6% −	20% +	1% −	61% −	38% −	29% −	63% −	15% −	.....

in the raw water was nearly five times as large from March 20th to 25th, when the more favorable results were obtained, than it was during the period of less favorable results, from May 1st to 6th; but the principal cause is due to the sample of filtered water of May 4th having contained a larger number of bacteria than was found in any

sample during the test, viz., 61, and as the sample of raw water of this date contained but 298, the percentage of reduction was only 78.89; which, being averaged up with the percentage of the other five days, brought down the total average of the special test, from May 1st to 6th inclusive, to 94.03 per cent.

The results given in Tables Nos. 1, 2, 3 and 4, are remarkably satisfactory; but on account of several instances in Table No. 1, when the bacterial reductions are shown to be below 98%, it may be well to state that the filter has been working at a disadvantage for a comparison by percentages, on account of the small number of bacteria in the raw water.

TABLE No. 3.—Color of Samples. Determined by Dr. Gardner T. Swarts.

Results obtained during the time that 1 grain of sulphate of alumina per gallon was used.

Rate of filtration, 125 000 000 galls. per acre per twenty-four hours.

Date.	Raw water.	Filtered water.	Date.	Raw water.	Filtered water.	Date.	Raw water.	Filtered water.	Percentage of color removed.
Mar. 13	0.30	0.06	Apr. 12	0.50	0.10	May 11	1.00	0.10	.....
" 14	0.30	0.06	" 14	0.50	0.10	" 12	1.00	0.10	.....
" 15	Sterilized filter-bed.		" 15	0.50	0.10	" 13	0.90	0.10	.....
			" 17	0.50	0.10	" 15	0.80	0.10	.....
" 16	0.30	0.06	" 18	0.60	0.10	" 16	0.90	0.10	.....
" 17	0.30	0.06	" 19	0.60	0.10	" 17	0.80	0.10	.....
" 18	0.30	0.06	" 20	0.60	0.10	" 18	0.80	0.10	.....
" 31	0.40	0.10	" 21	0.60	0.10	" 19	0.70	0.10	.....
Apr. 1	0.40	0.10	" 22	0.60	0.10	" 20	0.70	0.10	.....
" 3	0.40	0.10	" 24	0.60	0.10	" 22	0.70	0.10	.....
" 4	0.40	0.10	" 25	0.60	0.10	" 23	0.60	0.10	.....
" 5	0.40	0.10	" 26	0.60	0.10	" 24	0.60	0.10	.....
" 6	0.40	0.10	" 27	0.70	0.10	" 25	0.70	0.10	.....
" 7	0.40	0.10	" 28	0.70	0.10	" 26	0.60	0.10	.....
" 8	0.40	0.10	" 29	0.60	0.10	" 27	0.60	0.10	.....
" 10	0.50	0.10	May 8	0.70	0.10	" 29	0.60	0.10	.....
" 11	0.40	0.10	" 9	0.70	0.10	" 30	0.60	0.10	.....
" 12	0.50	0.10	" 10	0.70	0.20	" 31	0.50	0.10	.....
						Average	0.58	0.10	83.0

As can be seen by Table No. 1, the average number of bacteria found in the filtered water is less than 5 per cubic centimeter, which is a remarkably small number (the average number found in the effluent of the filter of the Lawrence water-works during March and April, 1898, being 28). Therefore, if the results given in Table No. 1

should be used for comparison with results obtained with other filters, the small number of bacteria found in the filtered water should be duly taken into consideration, as well as the percentages of reduction.

TABLE No. 4.—Alkalinity of Samples. Determined by Dr. Gardner T. Swarts.

Results obtained during the time that 1 grain of sulphate of alumina per gallon was used.

Rate of filtration, 125 000 000 galls. per acre per 24 hours. The alkalinity is expressed as calcium carbonate in parts per 1 000 000.

Date.	Raw water.	Filtered water.	Date.	Raw water.	Filtered water.	Date.	Raw water.	Filtered water.
Mar. 13	5.5	1.7	Apr. 13	10.2	Lost	May 11	14.0	6.0
" 14	7.5	2.0	" 14	11.5	4.0	" 12	14.5	6.5
" 15	Sterilized filter-beds		" 15	11.5	5.0	" 13	14.5	5.5
			" 17	12.0	4.5	" 15	15.0	5.5
" 16	6.7	1.5	" 18	11.0	4.0	" 16	14.0	5.0
" 17	7.0	2.0	" 19	11.0	4.5	" 17	14.5	6.5
" 18	6.7	2.0	" 20	12.0	2.0	" 18	14.5	6.0
" 31	9.0	2.7	" 21	11.0	4.5	" 19	14.0	6.0
Apr. 1	8.5	3.0	" 22	8.5	4.7	" 20	14.5	6.0
" 3	9.2	2.7	" 24	14.0	7.0	" 22	14.0	8.0
" 4	9.5	3.2	" 25	14.0	7.0	" 23	14.0	6.0
" 5	8.7	3.2	" 26	14.0	6.0	" 24	14.0	7.0
" 6	6.5	3.0	" 27	14.0	6.0	" 25	14.0	7.0
" 7	11.0	3.7	" 28	14.5	6.0	" 26	15.0	6.0
" 8	10.0	3.2	" 29	13.5	6.0	" 27	14.5	5.0
" 10	9.0	3.7	May 8	15.0	6.5	" 29	14.0	5.0
" 11	10.5	3.2	" 9	14.5	6.0	" 30	14.0	5.0
" 12	11.0	4.0	" 10	15.0	6.0	" 31	14.5	6.0

TABLE No. 5.—Results of Special Experiments.

Date. (Inclusive.)	Sulphate of alumina used. Grains per gallon.	Number of Bacteria.		Percentage less bacteria in filtered than in raw water.	Percentage less color in filtered than in raw water.
		In raw water.	In filtered water.		
March 20th to 25th.	0.75	1 768	28	98.32	79
March 27th to 30th.	0.60	875	9	98.75	73
May 1st to 6th.....	0.75	360	19	94.03	75
June 1st to 7th.....	1.00*	640	15	96.82	..

* Sulphate of alumina of low grade, containing 17.53% of $Al_2 O_3$.

Experiments have shown that some species of bacteria will multiply even in distilled water that has been sterilized; and it is quite possible that, should sterilized water be applied to the filter instead of the raw water, a few bacteria might be found in the effluent as it flowed from the filter, as it would probably be impossible to keep any practical

filtering medium, or water which had been exposed to the atmosphere, completely sterile during the process of filtration.

The disadvantage under which the filter has been working is in accordance with the supposition that a few bacteria may, at times, grow in a filter and be carried through it during the process of filtration. These few bacteria would, in ordinary practice, be counted among others found, if there were such, in the filtered water. Now, if there were 2 000 bacteria per cubic centimeter in the raw water, and a small number per cubic centimeter should grow in the filter, they might not appreciably affect the percentage of reduction, owing to their number being relatively very small in comparison with the number in the raw water. If, however, the number of bacteria that grew in the filter was the same, and there should be 200 or 300 per cubic centimeter, for instance, in the raw water, instead of 2 000, the percentage of reduction might be affected considerably, as the small number that grew in the filter might bear an appreciable proportion to the 200 or 300 in the raw water.

The cost of operating the filter since it was first put in service has been practically the cost of the sulphate of alumina used, as no additional labor has been required, other than that already employed at the pumping station. The cost of operating, therefore, based upon 1 grain per gallon, and, considering the best grade of sulphate of alumina used during the test, would be $2.15 per 1 000 000 galls. of water filtered.

The total cost of the filter plant (shown on Plate IV) is estimated to be about $11 500. If the three additional filters, for which the filter house was designed, should be added, as has been mentioned previously, at the same cost per filter as the one which has been installed, the cost of the completed plant, representing a capacity of 2 000 000 galls. per 24 hours, would be about $21 000, or at the rate of $10 500 per 1 000 000 galls.

The East Providence Water Company is more than satisfied with the filter plant, and the customers of the company are much pleased with the filtered water, the appearance of which is practically the same as that of distilled water.

DISCUSSION.

GARDNER S. WILLIAMS, Assoc. M. Am. Soc. C. E. (by letter).—It would be interesting to learn whether the samples for analysis were taken from the filtered-water well shown in Fig. 2 of Plate IV, or from a tap above the floor. Judging from the illustration it seems hardly probable that if taken from the latter they would represent the condition of the water supplied to the consumers, and if they were taken from the well it does not seem possible that they can represent the conditions for any great length of time. Mr. Williams.

Many know the ease with which dust and filth, not to mention liquids, will pass through timber floors. From the drawings or the description of the plant, there does not appear to be any provision made for preventing the passage of such accumulations, as well as the coagulant that might be spilled upon the floor, through into the filtered water. It would be surprising to learn that this important point has been really overlooked, but if it has, the filtration of this supply must be one of the greatest farces of which we have had a record, however well satisfied the consumers may be with the water furnished.

GEORGE W. FULLER, Assoc. M. Am. Soc. C. E.—This paper is an interesting one, in that it adds to the meager data now available with reference to the purification, by this method, of waters which are soft and also at times quite highly colored. Owing to the fact that the views of water-works men are not well crystallized as to the best method of treatment for waters of this type, the subject is an important one for discussion. Mr. Fuller.

In connection with the results of this test, as presented in the paper, there are a number of points upon which it is desired to make comments and inquiries, as follows:

Color of the Raw Water.—One of the most important lines of information, from a practical standpoint, in connection with the purification of water of this type, is to have a reliable record of the color which it contains, due to organic matters dissolved in it. These data are as requisite for water of this class as are records of turbidity and amounts of suspended matter in the silt and clay-bearing waters of Western rivers. With this method of purification, color records bear directly upon the amount of sulphate of alumina required; the margin of alkalinity; the required provisions for coagulation and subsidence; and the percentage of wash water.

The paper does not state the method by which the recorded results were obtained; but, in a pamphlet issued recently by the New York Filter Manufacturing Company on this same subject, it is noted that the color results were obtained by the Nesslerized ammonia scale. The

Mr. Fuller. speaker thinks that the use of this scale is unfortunate for this line of work, for the reason that it is not a progressive one, and that the amounts of color for successive tenths on this scale are quite variable. That is to say, the individuality of the scale obscures the relationship between color and several factors of practical value, as noted above.

The question of color standards has been investigated thoroughly at several places, and was discussed fully by Desmond FitzGerald, President, Am. Soc. C. E., in the 1893 report of the Boston Water Board. For this line of work the platinum-cobalt standard is much more desirable for yielding data of practical value. A comparison of the two color scales, according to Mr. FitzGerald, is as follows:

Nesslerized Ammonia Scale.....	0.10,	0.20,	0.30,	0.40,	0.50,	0.60,	0.70,	0.80,	0.90,	1.00
Equivalent on Platinum Scale....	0.18,	0.26,	0.33,	0.39,	0.46,	0.52,	0.58,	0.63,	0.70,	0.81

From this table it is seen that according to the more consistent platinum scale the recorded results for the raw water are too high, and for the filtered water they are too low.

As color refers properly only to dissolved matters, the results would also be abnormally high in the raw water if it possessed turbidity which was not removed prior to reading.

Relations between Color and Required Sulphate of Alumina.—The available evidence indicates that, when the raw water contains more color than is removed with the use of 0.75 to 1.00 grain of sulphate of alumina per gallon, it is necessary to add this chemical in the approximate ratio of about 2 grains per gallon for 1 part of color in the raw water when the latter is expressed on the platinum scale. Sulphate of alumina in this case would be the ordinary grade, containing approximately 17% of alumina soluble in water, or having about 80% of the strength of the grade used in the East Providence test.

Keeping in mind the relationship expressed above, there are found, upon studying the data recorded in this test, two points upon which the speaker would be pleased to obtain further information, as follows:

1. As the amount of coagulating chemical was applied to the raw water regardless of whether the latter contained 0.3 or 1.0 part of color on the Nessler scale, the indications are that there was considerable waste of chemical in the former instance.

2. The practically complete removal of color when it was highest in the raw water by 1 grain of sulphate of alumina appears to be decidedly out of line with what little evidence there is available upon this subject.

Alkalinity.—Here again the paper does not state by what method the analytical results were obtained, but from a recent article in *The Engineering Record* it is learned that they were obtained with the aid of methyl orange as an indicator. This procedure, as applied to this particular test, is a very questionable one, in that this method, even

when the composition of the indicator used is such as to make it as sensitive as possible, is incapable of showing the presence of undecomposed sulphate of alumina in the filtered water. In fact, experience shows that if the filtered water should contain several grains per gallon of undecomposed coagulating chemical it is quite probable that with this indicator the filtered water would be recorded as slightly alkaline, notwithstanding the fact that it would be markedly acid if properly tested by reliable indicators, such as lackmoid and erythrosine. Mr. Fuller.

With the grade of sulphate of alumina used in this test it is probable that on an average about 9 parts of alkalinity are required to decompose 1 grain per gallon of the applied chemical. Turning now to the table of alkalinity results as recorded in this paper, it is noted that for a period of several weeks the raw water ordinarily contained rather less than this required amount when treated with 1 grain. From this evidence it would appear very probable that the filtered water contained undecomposed sulphate of alumina in small amounts with considerable regularity at that time.

Relation between Color and Alkalinity of Raw Water.—In connection with the purification of water of this type, it is of especial importance to ascertain the minimum alkalinity of the raw water and the color which it possesses at such times, and also the amount of alkalinity at times when the water is colored most highly. By this means alone can it be foretold definitely what the probable chances are of meeting the inadmissible state of affairs of having undecomposed coagulating chemical in the filtered water, and whether or not it is necessary at times to resort to the practice of making applications of alkali to the water during treatment.

At East Providence the data are very meager in this respect, as they are in almost every other instance of soft, colored waters, within the speaker's knowledge.

Chemical Results.—The principal point in connection with the results of the ordinary chemical analyses of water before and after filtration is the reduction of the nitrogen in the forms of free and albuminoid ammonia, which, as found elsewhere, is seen to be quite marked; and leaving in all probability only such organic matter in the filtered water as is so stable in its composition that it has practically no significance from a hygienic standpoint.

The reduction in the total solids and in the ferric and aluminic oxides is explained normally by the removal of the silt and organic matter of the raw water. Concerning the determination of aluminic oxide, there is very little reason to believe that it throws light upon the presence of undecomposed chemical in the effluent.

The other determinations, as recorded, ought not to show any difference in the results between the raw and filtered water. The

Mr. Fuller. increase in total hardness is abnormal, as ordinarily the reduction in carbonate hardness corresponds exactly to the increase in sulphate hardness, thus giving the same total.

Bacterial Results.—It will be noted that the numbers of bacteria in the filtered water are very low indeed, as judged by evidence from other filter plants, and this is of course explained very fully by the thorough coagulation of the raw water, which ordinarily must have been obtained with the liberal amount of sulphate of alumina which was used. In this connection, the speaker would like to inquire as to the reaction of the culture medium which was used, and also the temperature at which the bacteria were cultivated.

Growths of Bacteria.—From page 78 one would gather the idea that it is suggested that growths of bacteria within the filter might explain in part the presence of some of the very few bacteria which the filtered water contained. The evidence indicates that this idea, in connection with the East Providence results, has very little to substantiate it, for the following principal reasons :

1. The opportunities for growth in the sand layer of a mechanical filter are very much less than in the sand layer of an English filter, because, ordinarily, the former is washed and agitated at least once a day, removing thereby the majority of the bacteria which are attached to the sand grains, and especially those bacteria which are most favorably located with reference to food supply.

2. While it is true that the bacterial results of the effluents of English filters frequently show to quite a marked degree the results of bacterial growth within the sand layer, the fact that in this particular mechanical filter the rate of filtration was something like 50 times as great as is ordinarily the case in English filters would cause such a dilution of the few bacteria which might possibly grow on the sand, that it would appear to cause this factor to drop out of comparative significance in the connection in which it is used.

Wash Water.—The records of this test, as given in the paper, contain no statement with reference to the frequency of washing the sand layer, and the amount of water required for that purpose. This is an important matter in the test of a mechanical filter, and especially so where it is used with a fairly high-colored water, and in which there is provided only 17 minutes' time as an average period of coagulation and subsidence prior to the entrance of the water into the filter proper. The speaker would like to inquire if there are any data upon this topic.

Automatic Controller.—The automatic controller described is one of the most interesting features of the plant from a mechanical standpoint, and it would be interesting to know what head it requires for its normal operation, and what the pressure of the filtered water upon it was at the end of the run, where it is recorded that the variations in the rate were less than one-half of 1 per cent.

Cost of Operation.—It would seem that in this connection the item of wash water should appear. Obviously it had some value as applied under pressure at the bottom of the sand layer in connection with the agitation of the latter. Mr. Fuller.

With a filter of this type the cost of labor, in efficient operation, is ordinarily a considerable item. Although no additional labor may be required in this particular instance, it would seem only reasonable that there should be a *pro rata* charge against this item, as the filter plant doubtless increases materially the amount of work for the pumping station attendants.

E. SHERMAN GOULD, M. Am. Soc. C. E.—The question of filtration of water is becoming a very important one, upon which no hydraulic engineer at present can afford to be uninformed. The speaker has been very much interested in this paper and in Mr. Fuller's remarks, and he thinks there are a number of points which could, with great interest, be elucidated in regard to this question. As between the slow, sand filtration and mechanical filtration, the prevailing idea seems to be that the former is preferable; that that is the standard method of filtration. But the great area which is required for the filtration of a large supply of water, and the necessity which seems to exist in the climate of the greater part of the United States for covering these filters, makes the proposition somewhat formidable. If anything quicker, smaller and cheaper can be devised, it will be a very great benefit, and, as far as the speaker can judge from his reading of what is going on throughout the country, mechanical filtration seems to be coming very much into prominence on this very account. In recent reports by a very high authority, upon the filtration of certain supplies to which the speaker's attention has been called, it is stated that exactly the same purification would be obtained whichever system was used. If that can be established as a fact, the mechanical system of filtration certainly has much to recommend it. Mr. Gould.

The speaker would like to be informed on one point which would seem to constitute a possible superiority of the mechanical filter. Although, in the slow sand filtration, by far the greater part of the purification is effected on the upper quarter-inch or half-inch of the fine sand, and in the gelatinous film which is formed by the water itself, that the lower courses of the filter, the coarser sand, the gravel, the coarse gravel, and through all the gradations, do exercise some refining and purifying influence on the water. Therefore, if those beds toward the bottom filter the water to some extent, they must retain some of its impurities. Now, as far as the speaker knows, the cleaning of a sand filter is limited entirely to the removal of the upper quarter-inch, half-inch, or inch, or whatever it may be, of the top surface, and he has not yet seen any account of the removal, cleaning and replacing of the entire filtering material, whether it be 3, 3½ or 4 ft. deep. If all the

Mr. Gould. filter is filtering, and thereby retaining impurities in greater or less quantity, it would seem that all the material should be renewed occasionally. But, when the mechanical filter is washed, all the material is washed; the whole filter from top to bottom is cleaned, and that might be a point of superiority. Is the period during which a sand filter bed is laid off for the purpose of having the surface scraped long enough to accomplish the purification of the rest of the filter by aeration? Has it ever been found necessary in sand filters to remove occasionally the entire material and clean it? If so, it must increase greatly the cost of operation and maintenance.

Mr. Currier. CHARLES G. CURRIER, Assoc. Am. Soc. C. E.—Filtration of water on a large scale is gradually becoming a necessity for various of our communities, and the skilled hydrologist knows how to recognize this better than politicians. Many a supply, which appears excellent to those who judge merely by the clearness, the taste, the color, and even by the results of chemical testing of a few samples, is regularly or irregularly a means of bringing disease to a varying proportion of those who drink it; while a supply may be highly colored and contain much "impurity" and still be harmless, as, for instance, that from the "Dismal Swamp," which years of experience have shown to be wholesome. The Croton water-shed yields an excellent water if we judge it by the mortality statistics of New York City, although it seems somewhat bad at times to citizens or visitors who happen to see vegetable detritus which appears at certain seasons.

So, too, an expert would condemn it if any considerable area of the water-shed was so much like a sewer and receptacle for refuse as the tributary stream which flows through Brewsters. The beneficent processes of Nature, operative during the interval after the water leaves this region of fouling and in storage and flowing gently onward diluted by more wholesome water, serve to minimize greatly the danger coming from contamination there. Yet sanitary engineering work is needed there and elsewhere, and public filtration of the entire Croton supply would make it more satisfactory, although other cities stand more in need of the process.

Sand filters have been in use in England for more than half a century. Until about sixteen years ago they were not recognized as having other than the obvious merit of clarifying water and producing some chemical improvement in it. At Berlin, where this method had been introduced in 1856, extensive new beds were constructed in 1883, for the Tegel river-lake supply, to strain out the growths of *crenothrix* which, owing to the presence of iron in the supply, developed so extensively as to obstruct the pipes. At that time the recently developed Koch nutrient-gelatine method of testing for bacteria was tried and soon revealed the fact that such filters, when working properly, had the great hygienic merit of holding back almost all the bacteria,

besides all visible impurities which abounded in the crude supply. This admirable result was found to be due to the dense film of minute vegetation, silt and other fine sediment which is caused to settle upon the top of the freshly cleaned upper layer of fine sand before any water is allowed to filter through. Some chemical improvement also takes place especially in the upper portion as the water passes through, though Frankland classes this as "slight," and, from the point of view of disease-prevention, this is wholly unimportant as compared with the practically complete arrest of any bacteria capable of inducing typhoid fever or other water-borne disease. * Mr. Currier.

It should be stated that—despite a number of special methods for isolating typhoid bacteria—it is nowadays not regarded as practicable, regularly, to detect these bacteria in water; and the advertisers who, after examination of a given sample of water, assure people that their supply is free from the germs of typhoid, ought not to be relied upon.

As with the very few varieties of domestic filters, such as the "Berkefeld" and the "Pasteur," which are capable of rendering a water germ free, so with all other filters for rendering water fit to drink, the essential quality is the mechanical straining action produced by a dense obstacle between the crude supply and the wholesome effluent. In sand filters the continuous surface film must, as above indicated, be produced and maintained perfect and unbroken. Otherwise, the product is unfit for use in case there be any disease germs in the crude supply. For "mechanical" filters, in all of which, like that described by Mr. Weston, the water flows through a relatively small bed of sand fifty to sixty times as fast as is the safe rule with the slow, gravity sand beds, and which are cleansed and renew their films many times, while the slow filters are making a single run of days or weeks, it is necessary to add a suitable coagulent, such as iron salts, or, more commonly, alum sulphate. One part of this latter salt, or even a little more, to 100 000 parts of crude water is usually added during the entire flow, and, to insure a satisfactory film, most operatives add considerably more than this to the first water let in at the beginning after each washing. In reply to the question as to whether the very minute amount of undecomposed alum, which is present in the effluent of a successfully managed mechanical filter, is in any way detrimental to health, or objectionable in any industry, it may be said that no sufficient evidence can be adduced against the careful use of alum in this way.

In testing numbers of different makes of mechanical filters under various conditions during the last decade, the speaker has found them to vary considerably in efficiency. When they were well managed and not overworked they were found capable of yielding results compar-

* *Transactions*, Am. Soc. C. E., Vol. xxiv, pages 40 to 58.

Mr. Currier. able with those of the best slow gravity sand beds. But altogether too often the operatives are careless, even if the plant be adequate and the appointments good. When not at their best they are inferior to good gravity beds. When yielding regularly such an excellent effluent as is represented by Mr. Weston's figures, even though the cost of operating be considerably greater than he indicates, they would be preferable to the large, slow sand-bed filters in places where suitable location for the latter type is not available, or where fine sand of the right sort is not readily obtainable at moderate cost. The usually lesser first cost of the plant, as compared with that of the large, slow beds, when well constructed, is an influencing element. Against them may be urged that their constant efficiency depends too much upon the persons operating them, and that defects in the hygienic quality of the effluent cannot be detected until that has got beyond reach in a general reservoir, if not already consumed. For each properly constructed slow gravity bed, on the other hand, separate storage basins are now in favor, and these can retain the effluent of a given filter bed until the usual test has shown whether the water is to be used or wasted.

As Mr. Fuller has remarked, the increase of one-fifth in hardness shown in the paper is contrary to the rule and must depend on some element not explained therein. The extent of bacteriological purification is notable. In answer to one of the queries it is proper to state that the very competent gentleman who tested for bacteria, in all probability used the familiar Koch's gelatine culture test, and his experience and reliability should cause one to give full credence to his report. When for three months of constant practical use of the filter the effluent reveals to this test less than ten bacteria for every thousand which were in the crude water, that is a very good showing. Yet the best slow gravity beds would be likely to show less fluctuation than is evident from the latter half of Table No. 3 and in Table No. 5 (May 1st to 6th). It is fair to assume that this same filter would have developed a still higher ratio of efficiency if in the same crude water the number of bacteria had been as high as occurs in some other unfiltered waters.

In considering water-filtration results, too much relative importance is still attached to the "chemical purification." The presence of ammonia in the filtrate does not in itself mean any menace to perfect wholesomeness. Good distilled water may show a far greater proportion of this than occurs in any natural supply. So, too, near the sea coast or in a saline region chlorine is to be expected in water supplies to a degree in excess of the average amount elsewhere. But it is hygienically of absolutely no importance that this is found to be reduced 1% or even much more in the effluent of this or other excellently working filters. If the iron is lessened three-fifths and the color more than four-fifths (tested in any constant, acceptable way) and the

water thereby rendered more acceptable, that, of course, appeals to the users and is in so far a benefit. But what they do not appreciate, and what is nevertheless of paramount importance, is the immense amount of bacteria kept out of the purified supply by the best filters, and with which any chance disease germs are also held back and annihilated. Some waters, apparently quite wholesome, have a considerable number of bacteria, and on the other hand, waters can be infected with the germs of typhoid fever and other diseases and yet have a relatively low number of bacteria of all kinds. It is not of itself the mere quantity so much as the nature of the bacteria present which makes the element of positive danger in a drinking water. Since the separation of practically all the bacteria of a bad or questionable water supply means also the removal of all germs of disease, that should be the ideal and practical result always aimed at. If with this an exceptional degree of chemical purification is achieved, so much the better. Inasmuch as the nutrient-gelatine culture test is simple and easy to carry out by careful people, even if they be not at all familiar with laboratory technique, that valuable test should be used regularly in a uniform way in connection with the operation of all filters. Mr. Currier.

EDMUND B. WESTON, M. Am. Soc. C. E. (by letter).—The writer has read Mr. Fuller's discussion with a great deal of interest. Mr. Weston.

The color data given in Table No. 3, from March 13th to 18th, inclusive (5 days), as originally furnished the writer, were incorrect. Instead of 0.30 for the raw water and 0.06 for the filtered water, the figures for the 5 days should read 0.50 for the raw water, and 0.10 for the filtered water.

The writer, of course, did not propose to do chemical work in regard to the subject in hand. As to chemical matters, he is aware that different chemists sometimes hold different views on a given subject.

The chemical work was referred to Professor John H. Appleton, of Brown University, a gentleman whose age, long experience and conservative judgment entitle his opinions to great weight; and the writer still feels entire confidence in the analytical work, and the opinions expressed by Professor Appleton in the chemical questions involved in this discussion.

Color of the Raw Water.—Mr. Fuller remarks that the paper "does not state the method by which the recorded results" on color were obtained. They were obtained by the platinum-cobalt method.

Mr. Fuller remarks that "in a pamphlet issued recently by the New York Filter Manufacturing Company * * * , it is noted that the color results were obtained by the Nesslerized ammonia scale." Mr. Fuller misunderstands the pamphlet. The pamphlet states what "the unit of color" is; but does not state the "method." The platinum-cobalt method was used, but the unit of color of this method

Mr. Weston. is, as the pamphlet states, "practically that color yielded by properly Nesslerizing 50 c. c. of water containing one-hundredth of a milligram of ammonia gas (or its equivalent)." It appears, therefore, that Mr. Fuller's inferences with respect to the color data are not in accord with the facts.

Relation between Color and Required Sulphate of Alumina.—It was desired, if the alkalinity of the water would permit, to obtain a bacterial removal of at least 99% without regard to the quantity of sulphate of alumina used. It was found that 1 grain of sulphate of alumina per gallon would do this without rendering the filtered water acid; and it was also found that when $\frac{3}{4}$ of a grain was used, the bacterial removal was less than 99 per cent. It was therefore decided to use 1 grain.

As to Mr. Fuller's inference that there was a considerable waste of chemical at times, Mr. Fuller probably intended the statement to be taken in a relative sense, as the whole amount of sulphate of alumina used daily was not considerable, averaging less than 28 lbs., the daily cost being about 46 cents.

It should be remembered that the filter was in practical service during the test, and was not being run as a laboratory experiment; and as the bacterial removal averaged more than 99%, and the color of the filtered water was hardly distinguishable from freshly distilled water, and was sufficiently alkaline to show that the quantity of sulphate of alumina was being kept within the proper limit, it would hardly have been practicable, even if a small quantity of the sulphate could have been saved, to have made, from time to time, minute changes in its amount, as the cost of the labor of doing so would have been of much more account than the cost of the sulphate of alumina which might have been saved. Then, if this refinement had been gone into, and an experienced person had been employed to have continually kept the run of the alkalinity of the water, the expense would have been many times greater than the cost of the whole of the sulphate of alumina used.

It would appear as though Mr. Fuller's inferences from his experience with the water of western rivers would not apply to river waters in the vicinity of Providence.

During the Providence filtration experiments, in 1893 and 1894, it was demonstrated that the percentage of color removed from the raw water could not be relied upon as a gauge, in respect to the removal of bacteria, and the results of the East Providence test show the same to be the case.

At East Providence, as has previously been stated, the paramount desire was to remove at least 99% of the bacteria from the raw water, provided that it could be done without exhausting the alkalinity of the water and causing the filtered water to be acid, the importance

of the removal of the color from the raw water being regarded as secondary to that of the bacteria. Mr. Weston.

As the bacterial results for each day were not known until about five days afterward, on account of the time required for cultivation, it would not have been possible to have gauged accurately the quantity of sulphate of alumina, more or less, which might have been the most advantageous to have used each day; therefore, as experience had shown that $\frac{3}{4}$ of a grain of sulphate of alumina would not produce an average bacterial removal of 99%, and that 1 grain would accomplish the desired result, and as the filtered water was always alkaline when 1 grain was used, it was thought that the constant use of 1 grain per gallon was the most satisfactory method of applying the sulphate.

Alkalinity.—Mr. Fuller makes a considerale body of comments on the alkalinity of East Providence water. The writer discusses these comments briefly:

The alkalinity determinations were made as follows: two portions, each of 500 c. c., of the water were placed in flat white porcelain trays side by side. To each sample, 5 c. c. of solution of methyl orange was added. First one sample and then the other was titrated with standard sulphuric acid, the acid being so prepared that each cubic centimeter would neutralize 1 part per million of calcium carbonate in 500 c. c. of water. (The sulphuric acid solution was standardized by pure sodium carbonate; then its value in calcium carbonate was computed.)

Mr. Fuller objects to methyl orange as an indicator. The writer must rely on Professor Appleton's statement that methyl orange is, in fact, a sensitive indicator for acid and alkali, that it is widely used for this purpose, and is recommended by high authorities on water analysis. Indeed, it was used during the elaborate filtration experiments conducted under the direction of Allen Hazen, Assoc. M. Am. Soc. C. E., at Pittsburg, Pa.; and Mr. Hazen appears to have been entirely satisfied with the reliability of the alkalinity determinations made with methyl orange.

From certain experiments made elsewhere by Mr. Fuller, he forms the opinion that the filtered East Providence water, during a portion of the test, must necessarily have been acid. But this is an opinion. As the result of actual tests, the filtered water was alkaline. That is, a considerable quantity of the standard sulphuric acid was necessary to overcome its alkalinity.

Mr. Fuller states that, in his opinion, a filtered water might "contain several grains per gallon of undecomposed" sulphate of alumina, and yet that such water might be slightly alkaline to methyl orange. In the East Providence filtered water there could not possibly have been several grains per gallon of undecomposed sulphate of alumina, since not more than 1 grain was added to the raw water.

Mr. Weston. *Chemical Results.*—Mr. Fuller appears to represent that the analytical determinations of alumina, Al_2O_3, in the raw and the filtered waters are of little account. The writer holds the opposite view. He thinks them of considerable importance. They certainly show that in all cases the amount of alumina, Al_2O_3, in the filtered water was very small. It varied from about 0.02 to about 0.06 of a grain per gallon. But the 1 grain of sulphate of alumina added in the coagulant contained 0.22 of a grain of alumina, Al_2O_3. It is plain, therefore, that at least a considerable part of the alumina, Al_2O_3, contained in the sulphate of alumina applied, was removed by the process of filtration. Then, again, the analytical determinations show that there was an average of 38% less alumina, in the filtered water, than in the raw water before the sulphate of alumina was added to it. The writer considers these interesting and important facts.

Bacterial Results.—The culture medium used was 10% gelatine, and the reaction was slightly alkaline. The bacteria were cultivated at the average refrigerator temperature, the temperature of the laboratory being high at all times.

Growths of Bacteria.—The writer fears that he did not make his possible solution sufficiently clear, and that Mr. Fuller has interpreted his intent rather too broadly. It occurred to the writer, upon three or four occasions when the number of the bacteria in the filtered water had increased in a much greater proportion than those in the raw water, that it might have been due to a few bacteria growing in the filter. It was not his intention, by any manner of means, to even suggest the inference that bacteria ordinarily propagate in mechanical filters as they do in slow sand filters.

Wash Water.—From records kept during March, April and May, while 1 grain of sulphate of alumina, containing about 22% of Al_2O_3, was being used, the average length of the runs of the filter, which is the period between washings, is shown to have been about 6.6 hours; the range being from about 5 to about 9 hours.

During these runs, the height of the surface of the water in the filter (which remains practically constant), was about 10.85 ft. above the surface of the water in the controller. The operating head, or the head consumed during the process of filtration, was about 10.25 ft., namely: the difference between the level of the water surface in the filter and the elevation of the water (corresponding to the head upon the inlet pipe of the controller), above the surface of the water in the controller at the time the filter was shut down for the purpose of being washed. Immediately after washing, at the commencement of a run, about 2.92 ft. of the operating head was lost by friction, due to the water passing through the clean filter-bed, screens and outlet pipes.

On account of the desirability of supplying the raw water to the

filter by gravity, the filter was made only 12 ft. high, which is 4 ft. less than the standard height of filters of the Jewell gravity type, of the capacity installed at East Providence. If the filter had been 16 ft. high, the standard height, the operating head would have been 4 ft. more than 10.25 ft., and, consequently, as the greater the operating head, other things being equal, the longer a filter will run, the average length of time between washings would have been longer than 6.6 hours. Mr. Weston.

As the filtered water is being pumped constantly from the filtered-water well into the mains, it would be inconvenient to measure the amount of wash water used each time the filter is washed. From several measurements which have been made, however, the indications are that the average quantity of wash water used does not exceed 4% of the total amount of water filtered.

Automatic Controller.—The operating head, during the test referred to by Mr. Fuller, was 9.35 ft., and the head upon the inlet pipe of the controller at the end of the test was equivalent to a height of 1.5 ft. above the water surface of the controller.

The preliminary tests of the class of controllers used at East Providence are made with heads ranging from 18 ft. above the surface of the water in the controller as a maximum, to 0.33 ft. as a minimum.

Cost of Operation.—The wash pump is driven by a water turbine wheel, of much greater power than is necessary, which had been installed for another purpose before the filter plant was contemplated, and as the East Providence Water Company owns the water privilege from which the water required to operate the turbine is derived, the writer hardly thinks it would be advisable for him to go to the expense of indicating the power required to drive the pump, although, as a matter of scientific interest, he would like to know what it is. He can state, however, that a test, made about two months ago, showed the maximum horse-power of the water pumped while washing the filter-bed to be about 14. The horse-power was computed by considering the maximum quantity of water pumped per minute, the water pressure at the discharge end of the pump, and the elevation of the pump above the water in the filtered-water well. If the filter had been 16 ft. high instead of 12 ft., the other conditions being equal, the horse-power would have been about 15.6.

The pumping engineer, who has charge of the filter plant, estimates that the cost of the labor required for taking care of it is about $0.50 per day. As the writer has already stated in the paper, no additional labor, other than that which was employed before the filter plant was built, is required to take care of the filter plant. This $0.50, considering the present consumption of about 200 000 galls. would equal, proportionately, $2.50 per 1 000 000 galls. Of course the cost per 1 000 000 galls. would be proportionately reduced if the filter was

Mr. Weston. running the entire 24 hours and delivering its full capacity of 506 000 galls., and it would be very much less per 1 000 000 galls. if the three other filters, for which the filter building was designed, were installed, and running at their full daily capacities.

In reply to Mr. Gould—the general practice in England appears to be, after the repeated scrapings of a filter-bed have reduced its depth to the minimum limit, to dig off the old sand in sections above the gravel and replace it with a layer of fresh or washed sand, the old sand then being filled in upon the clean sand.

The advantage of being able to sterilize the filter-beds of mechanical filters, the writer considers to be of much importance.

The writer appreciates highly the thorough manner in which Dr. Currier has treated the subject in his carefully prepared and instructive discussion.

All processes of filtration, to be successful, must have intelligent supervision. Professor Percy Frankland, whose connection with the London water companies is well known, states, in regard to slow sand filtration:

"But the responsibility which we have seen attaches to this treatment of water cannot be exaggerated, for whilst when efficiently pursued it forms a most important barrier to the dissemination of disease germs, the slightest imperfection in its manipulation is a constant menace during any epidemic."

Professor William P. Mason, of Troy, N. Y., has stated, in regard to the subject:

"A filter, of whatever type, is a more delicate piece of apparatus than is generally recognized, and it requires constant attention of the most careful kind. In the mechanical form of filter, this care must, of necessity, be constantly forthcoming, or the filter would not run a day; the English bed, on the other hand, may be, and to my knowledge is, at times grossly neglected, and that too where the volume of the supply would seem to call for more attentive supervision."

The samples of filtered water, to which Mr. Williams refers, were taken from the controller and not from the well.

It would seem, by the somewhat eccentric language used by Mr. Williams, that he has derived considerable satisfaction, in drawing from a small outline sketch some rather humorous inferences in regard to the construction of the filter building.

Vol. XLIII. JUNE. 1900.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

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No. 869.

THE REACTION BREAKWATER AS APPLIED TO THE IMPROVEMENT OF OCEAN BARS.

DISCUSSION.*

Mr. Wisner.

GEORGE Y. WISNER, M. Am. Soc. C. E. (by letter).—The ideal conditions for obtaining satisfactory results from a single curved jetty are usually found where the structure can be made a continuation of the natural curve of the outlet of the harbor entrance. The necessity of placing the jetty on the north side of the entrance at Aransas Pass makes a reverse curve of the channel, with a tendency to shoal at the "Crossing," and in order to maintain the full channel depth of 20 ft. between curves without dredging will probably require a spur jetty on the south side of the channel as far out as the wreck *Mary*.

Galveston Harbor, Cumberland Sound and Coatzacoalcos Harbor, Mexico, are good examples of natural conditions where a single curved jetty, properly located, would be certain to produce beneficial results. The Coatzacoalcas River is a silt-bearing stream, but the formation of the bar indicates that the load of sediment carried at times of floods is not sufficient to produce deposition from slight changes of velocity of current.

The formation of the west side of the entrance is also such that a single jetty constructed on the east side of the channel would practically control the river current in a restricted channel across the bar into deep water, and if the littoral current is from the eastward, as

* Continuation of the discussion on the paper by Lewis M. Haupt, M. Am. Soc. C. E., in *Transactions*, Am. Soc. C. E., Vol. xlii., p. 485.

Mr. Wisner. has been stated, a single curved jetty would be the proper remedy to apply.

It should be stated, relative to the fourth conclusion* that the limitation of the use of single jetties at the mouths of silt-bearing rivers is intended to apply only to such streams as the Mississippi and Brazos Rivers, where the load of sediment during freshets is such that any diminution of velocity of flow produces deposition.

The phenomena observed at the South Pass jetties, relative to the effect of curves and of outlets through jetties near the shore, have an interesting bearing on some of the conclusions brought out in the present discussion.

The Act of Congress, under which the South Pass jetties were constructed, requires the maintenance of a channel through the jetties 26 ft. in depth, not less than 200 ft. in width at the bottom, and having through it a central depth of 30 ft. without regard to width.

In 1888, the conditions in the Pass became such that, in order to maintain the legal channel, dredging was necessary during most of the time when such work could be done. The writer was employed to make the necessary improvements to prevent the periodic shoaling and narrowing of the channel, and, from a careful study of the situation, concluded that the deposit of sediment in the channel was due to breaks in the jetties, allowing a large amount of water to escape before reaching the end of the jetties, and that the curvature of the channel was such that excessive depths developed near the concave jetty and caused sufficient deposit on the convex side to reduce the width at a depth of 26 ft. to less than 200 ft.

The construction of spurs, or short wing-dams, along the face of the concave jetty at intervals of about 500 ft., checked the tendency of the convex bank to encroach on the channel, and the repairs of the breaks through the jetty walls stopped the excessive deposits, and fully justified the conclusion that the correct remedy had been applied.

During the construction of the jetties at the mouth of the Brazos River, a heavy freshet occurred when the east jetty was built above high water for its entire length, and the west jetty only about three-fifths of its final length, which resulted in scouring out a channel 25 to 30 ft. deep between the jetties, and built up the bar beyond the outer end of the west jetty, so that the depth was less than previous to the flood. Careful study of the phenomena at both the South Pass and Brazos channels indicates clearly that, unless the flood waters of the rivers are confined within the channels until discharged into the littoral current outside of the bar, navigable entrances cannot be maintained.

The amount of curvature given a channel fixes the width which can be maintained, and, if the curves be made sharp relatively to the

* See page 516, *Transactions*, Am. Soc. C. E., Vol. xlii.

volume of flow, sufficient width of channel cannot be maintained for the safe navigation of large vessels. Mr. Wisner.

Experience shows that deep channels are maintained easily along the concave sides of curves, and are deeper and nearer the bank as the curve is sharper.

The steepness and character of the banks also have much to do in fixing the depth and position of the deepest water relative to both sides of the channel. A steep smooth bank along the concave side of a curve, with a shore having a gentle incline toward the bed of the channel on the convex side, is best adapted for maintaining the ebb flow parallel to the concave bank with minimum curvature of the channel.

The results thus far obtained by the incomplete works at Aransas Pass indicate that the design and location of the structure is well adapted for developing such conditions.

J. Francis Le Baron, M. Am. Soc. C. E. (by letter).—The writer has been familiar with Professor Haupt's theories in regard to breakwaters since 1888, and is glad that an opportunity has arisen for testing them under such favorable circumstances. The writer uses the word favorable, because it seems to him that the conditions at Aransas Pass are peculiarly adapted to successful improvement with a single jetty, although while the old Government jetty remained, the harbor was effectively bottled up. Mr. Le Baron.

We have here at Aransas Pass a large sandbank on the windward side of the channel, which is unquestionably caused by the diagonal action of the waves under the strong Northers moving the sand down before them, until they reach the outflowing current of the Pass.

If this current were not there, the sands moving continuously, ever as a resultant of the diagonal waves, would speedily extend the shoal to Mustang Island and perfect the littoral cordon. Anyone who is familiar with the sea must have noticed that this diagonal action is present on nearly all beaches the greater part of the time. The reason is that, with the wind blowing from all points of the seaward semicircle, except the normal, and, perhaps, perfectly parallel to the littoral, or even a couple of points off shore, there are some 19 points of the compass which produce a resultant diagonal wave action with the ground swell always rolling in from the outer sea, which is operative in this case, not only whenever the prevailing winds blow, but with every sea breeze, except that which is dead on.

The marked effect of strong winds blowing diagonally to a long straight coast line is seen on the Florida east coast between Cape Canaveral and St. John's Bar.

With a stiff Norther blowing, the surface water is blown bodily along the beach inside the breakers to the southward; the broken and lumpy water offering a better hold for the wind, which creates a strong race in the shallow water. This results in piling up the water to the

Mr. Le Baron. south, and, to restore the equilibrium, a strong counter-current is established just outside the breakers; and so it happens that whenever a Norther is blowing there is always a stiff current running up the beach in the face of the wind, which is very noticeable. The writer has observed the same thing on the north coast of Honduras during easterly winds, which are the prevailing winds in that locality. He has measured the velocity there and found it to be 1.04 miles per hour inside the breakers. This current was heavily charged with sand which was being constantly hurried to the westward and deposited on the spit on the east side of the mouth of the Patuca River, which in this way had been built out 400 ft. in eight years above water. If this spit could be induced to extend itself some 2 700 ft. it would form a perfect jetty, and this is what the writer proposes to do; to assist Nature by building the lightest jetty possible along the extension of the spit, which lies below water, so as to prevent the water from the harbor overflowing the crest of the spit and the sand from windward driving into the harbor; the spit will then be protected in its work of completing the littoral cordon.

Now, this is what has happened at Aransas Pass, where conditions appear to be quite analogous with the work being done at Patuca Bar. The jetty built by Professor Haupt has arrested the drive of sand to the southward and has prevented the escape of water over the rim of the north bank. Its curved shape has undoubtedly assisted the cutting out of the channel, for the reasoning of Professor Haupt on this point is not only logical, but is borne out by facts as we find them in all natural water-courses having any current, where, as every truant school boy can tell you, the deepest water is found in the bends, where he goes fishing.

This is caused by the revolving motion of the water, caused by the current striking the concave bank at an acute angle, and being guided by it in a circular path causing the formation of whirlpools which suck up the sand of the bottom, in the same manner that everyone has noticed a miniature whirlwind on a hot day, suck up in its vortex the bits of paper and dust in the street.

Thus far, the writer agrees entirely with the author, but is forced to differ from him on several other points. One of these is the little weight he is disposed to give to physical surveys and examinations; and here is shown the fallacy of leaving a breach in his wall to admit the flood tide near the shore. While the writer agrees with the author fully in the great and well-nigh imperative importance of the study of comparative charts, he cannot conceive how any permanent improvement of an ocean bar can be intelligently studied without the help of all the physical data which it is possible to obtain. These data, as the author observes, may sometimes seem conflicting and are often confusing, but it is the province of the engineer to so study and group them

as to harmonize or eliminate the apparent discrepancies and contradictions, which, on careful scrutiny, will generally be found to arise from imperfect or improperly directed examinations, wrong locations of observing stations, or abnormal conditions arising from unusual phases of meteorological or fluvial regimen. Mr. Le Baron.

It also often happens, as in the work upon which the writer is now engaged, that no previous survey of the locality has ever been made.

If we are to depend only on comparative charts, an engineer could lay out a system of improvement without ever going on the ground in person, which the writer thinks few practical men would care to undertake; but instrumental surveys, which the author condemns, are only more careful and extended personal observations, elaborated too much, perhaps, at times, but still only a method of arriving at facts which a simple visit cannot fully determine. Unless we know the direction and intensity of all the forces operating on an ocean bar at different points, how are we to estimate the effect of these forces, after the changes produced by our proposed works?

The study of charts alone, without surveys to determine the intensity and direction of the forces operating, is also misleading, when studied in plan alone, as volume is apt to be overlooked.

It would seem that the author has fallen into this error when he says:

"The effect of the two jetties is to invert the natural trumpet-shaped opening, and to diminish the area of the gorge, which is transferred to the crest of the bar, thus reducing the tidal volumes, preventing the complete filling of the interior compartment," etc.

The author likens the natural river mouth to a trumpet. A more accurate comparison would be made by considering the trumpet to be flattened until it represented a cubical sector. This sector has a depth, let us say, of 4 or 5 ft. When we narrow the opening over the bar we simply turn the sector on its edge and thus gain in depth what we lose in width. The cross-sectional area is practically the same as before.

The author speaks of preventing the complete filling of the interior compartment of the Pass, by the construction of converging jetties. The writer would like to ask him if he has ever seen any river mouth or pass which has been improved by converging jetties, the interior compartment of which failed to fill. We know that this has been prophesied often, but the writer has yet to find a case where the interior compartment failed to fill. It was feared that something of this kind might happen on the St. John's River, Fla., where converging jetties have been built, and upon which the writer was employed for several years as United States Assistant Engineer, but careful surveys and examinations, made by the United States Engineers after the jetties had been built, failed to show any diminution of volume.

Mr. Le Baron.

It is admitted that jetties might be built so close together that almost no tide-water could enter, but in that case the river, or drainage, water from the surrounding water-shed would fill the compartment, which would be practically dammed up, with only a sluice-way between the jetties. This might happen while the works were in an unfinished state, before the jetty channel had acquired its contemplated normal depth, and the writer can see how we might easily raise the water level, but how we can lower it in the interior compartments he cannot see.

If we dredge and clear away all sand banks from the mouth of the river, or pass, and widen and deepen it, the low-water plane of the interior compartment may be lowered and the range of tide increased, but if we close up the mouth with a dam, which two jetties amount to, with a sluice-way between them, we will most certainly raise the water level in the interior compartments, and reduce the tidal range, just in proportion to the size of the sluice-way between the jetties. If, then, we raise the water level, we must have more water in the compartments, and the writer fails to see why fresh water is not just as good as salt water for navigation or for scouring.

Even where no river debouches through the jetties, and we have only a salt-water lagoon or harbor, the case would be rare indeed, where, taking a period of a year or several months, the effluent discharge did not exceed the influent. The reason for this is, that there is always a water-shed of greater or less extent to every harbor, and the writer ventures to say that Professor Haupt would never succeed in draining even a purely tidal basin by building a dam across it with an open sluice-way. Those who have had experience in draining tidal marshes know how difficult it is to accomplish this, even with tidal sluice-gates.

For these reasons the writer is utterly opposed to the plan of leaving an opening next the shore, or anywhere else, except in the jetty channel, for the admission of tide water. An opening through which the flood tide can enter permits more or less of the ebb tide to go out, and by just that much we lose scouring power in the ship channel, and are likely to set up a dangerous scour in the subsidiary channel, which may endanger the stability of the works, or bring an undesirable amount of sand into the harbor.

This problem is easily deduced by the *reductio ad absurdum*, for if a small channel is a good thing, a larger one is better. Then if we make it with the same cross-sectional area as our main channel, we will lose nearly if not quite the same amount of water, depending on the relative velocities, and so the scouring effect is reduced; for, in spite of the incident, quoted by the author, of the flood last summer on the Brazos not deepening the channel between the jetties at the mouth, we know that lessening the volume and velocity of discharge

produces bars and banks, which are swept away when floods come, and these cases are so well known and numerous as to require no demonstration. Mr. Le Baron.

If, then, instead of having two openings of say, the same width, with only 10 ft. depth each, we turn all the water through one of 20 ft. depth, the writer fails to see why the flood tide cannot find this opening and make as good use of it as is required. But the writer would prefer to keep all the flood tide out, if it were possible, and replace it with water of drainage or river water, even to the extent of creating a greater velocity between the jetties than might be desirable, which there would be some danger of doing if the mistake was made of making the channel too narrow.

This is a simple matter of computation, however, and it is the duty of the engineer to make it just right.

Leaving a jetty without any support, or with a gap between it and the shore, seems to the writer like sending a forlorn hope into an enemy's country unsupported. We are fighting the forces of Nature, and we must be careful and not be cut off from our reserves.

Another reason for leaving no gap in the line behind us is the ever-restless sand, which, as previously explained, is always traveling up or down the beach. If an opening is left here it deposits itself in the fair-way, but if the jetty is continuous to the shore the sand soon fills up the bight with a curved foreshore advanced to near the outer end of the jetty, backing and protecting the work, and rendering it indestructible by the waves.

For this reason, the jetty, in this position, can be very much less massive than would otherwise be necessary, and so the cost be reduced immensely and the river banks made continuous to the sea end.

This is what happened at the Suez Canal, Port Said; at St. Johns Bar, Fla., where the seaward angle of the south jetty filled up for over a quarter of a mile out; at Greytown jetty, Nicaragua; and what is sure to happen in every similar case on a sandy shore, where the prevailing winds blow along shore and into the bight made by the jetty.

The writer laid out the jetty at Greytown, making an angle of about 78° with the shore line to the eastward, from which quarter came the prevailing wind, and whence the sand was constantly moving along the beach. The location of this jetty was afterward moved quite a distance to the west, but the principle remains the same. In the writer's opinion, a large amount might have been saved in the cost of construction at Aransas Pass, if the jetty had been connected with the shore in the first place, as, although longer, it could have been much less massive and more secure, for Nature would then have been assisted in building up the foreshore, which would have formed a solid spit on the back or windward side.

Mr. Le Baron. While the jetty at Aransas Pass has proved a success in securing deeper water, the writer does not believe that we can make a hard-and-fast rule, applicable to every bar. Each case must be studied with the help of all the data which can be obtained, and the success in this instance certainly does not warrant the author's sweeping condemnation of convergent or parallel jetties, which have been successful, as the author admits, at the Danube and Mississippi Rivers, at Tampico and also at St. John's River, Fla., Charleston, Newburyport, Galveston, Brazos River, the Swinemunde Haff, Germany, Volusia Bar, Lake George, and undoubtedly would have been at Cumberland Sound, which the writer laid out for General Gillmore, had it not been for the peculiar management of the work by Captain Carter.

At St. John's Bar the depth has been increased from 12 to 22 ft. At Volusia Bar, of which work the writer was in charge, first as engineer and later as contractor, the depth was increased from 4½ to 6 ft., all that was demanded by boats of the class navigating it. Many others might be mentioned.

Where dredging has been resorted to in connection with these jetties it has generally been done while the works were yet in an uncompleted state and to hasten the development of a deep channel at the instance of some impatient board of trade or meddling member of Congress. Sometimes it is rendered necessary, owing to the location of the jetties and the resulting jetty channel, as at St. John's Bar and at Greytown, where the new channel, in both cases, had to be cut through a sand bank 4 and 5 ft. above the water. At St. John's Bar the south jetty crossed the main ship channel, in order to make the resulting jetty channel take a more direct route to deep water outside the bar, and this channel had to be made through a large sand bank, known as Ward's Bank, which was dry at high water and nearly 6 ft. above low water; otherwise the north jetty would have had to be more than double its present length. It is a question if it would not have been cheaper, in the end, to have followed the natural channel, in this case, and assisted Nature to deepen it. After all, in all bar improvements, the main thing is the location of the jetties, and it is always safest to follow and assist Nature, when possible. This is shown, in the case under consideration, by the location of the old Government jetty, which was a flagrant example of wrong location, as it was built to leeward of the proposed channel, leaving it entirely unprotected from the encroaching sands to windward, with the result that it failed entirely to produce the desired results.

The use of curved, instead of straight, jetties is not new, as is well known. Here we have two jetties, both curved, but one properly located and the other improperly. The old Government jetty is evidently an attempt to follow the plan adopted at Swinemunde Haff, below

Stettin, in Germany, where a curved jetty was built over 20 years ago, which has proved successful; but this jetty was built to windward of the channel, as it should be, whereas in the case at Aransas Pass the fatal mistake was made of building it to leeward. The fact that one jetty has been built here which has secured deep water does not prove that two jetties, if properly located, would not have produced the same, if not better, results; but if we can dispense with one of the jetties, and so save in cost, of course, it is preferable to do it. Mr. Le Baron.

The reason the author's jetty has succeeded and the Government jetty failed is, in the writer's opinion, entirely due to its location, and there is little doubt that a straight jetty in the same location would have produced the same results, as the main thing in this case was to protect the channel from the encroaching sands to windward. This done, Nature could safely be left to do the rest. In work of this kind we cannot cut out a pattern, lay it down on every chart of an ocean bar, and expect a jetty or breakwater built after it to open a channel, any more than we can take the locations, plans and profile of one railroad and use them for every other. Hardly any two cases can be treated exactly alike, but every one must be studied in detail, and with all the light of past experience and history to aid us in digesting and formulating all the facts obtainable in each particular case.

GARDNER S. WILLIAMS, M. Am. Soc. C. E. (by letter).—Although the writer can lay no claims to experience in the construction of jetties or the improvement of harbors, it has come in his way to study quite closely the action of water upon the surfaces which confine it, and it seems to him that the so-called reaction breakwater is the only practical application of the real forces of erosion in moving water which has been cited in the present discussion. Without intending to in any way show disrespect for the older members of the profession or wishing to be accused of partisanship, it must be said that the true and most effective cause of scour by water currents seems to have been almost entirely overlooked, even by so eminent an engineer as Captain Eads. Mr. Williams.

The flow of water in straight channels of regular cross-section is not likely to be accompanied by strong scouring action, even at quite high velocities, because the direction of the flow of the individual filaments of the water is tangential to the bounding surface, but let a curve of long radius be introduced and something quite different will occur. The point or region of maximum velocity will be disturbed and carried toward the convex side of the stream, and the resulting rearrangement of the velocities will produce something approaching a spiral motion in the water, which will not be for any considerable distance tangential to the bounding surfaces, and hence erosion will take place at once. If the curve be continued for a sufficient distance, the velocities rearrange themselves in conditions most closely conforming to their equilibrium and the scour diminishes, the direction

Mr. Williams. of the currents becoming again tangential. If, now, a tangent be introduced, the arrangement of velocities in the filaments is again disturbed and excessive erosion again sets in, to disappear once more when they have again arranged themselves. The success of the reaction breakwater lies in the fact that it has set up these non-tangential currents, and the success of many double straight parallel jetties lies in the circumstances of their having received a stream from a tortuous river channel and forced it to straighten itself as it flows through them, thereby setting up the cross currents; while the failure of others of similar construction has been due to the circumstance that they received a stream already moving in comparatively straight lines, and have continued it in the same lines without setting up non-tangential currents. So, we should not expect that in the improvement of tidal harbors the passing of a volume of water in and out between straight parallel walls would produce any very great amount of scouring; but if it be passed along a curved surface, in and out, a continual setting up of eddy currents will be produced and the scouring accomplished. The writer, only a few days ago, had an opportunity of observing a current full of eddies moving stones half as large as a man's head, while a few hundred feet further on, the same water, with a lineal velocity twice as great, would not move stones the size of hens' eggs.

Mr. Corthell. E. L. CORTHELL, M. Am. Soc. C. E. (by letter).—The last paragraph of the paper refers to the channel at the mouth of the Brazos River, Texas, in which it is stated that, contrary to the expectations of increased depths by the great flood of July last, the sounding showed but 15 ft. in several places, "thus demonstrating the relative superiority of the principle of reaction by a single jetty."

From a newspaper clipping it is learned that the United States Assistant Engineer in charge of the surveying expedition at the mouth of the Brazos River, has reported to Captain Riche, U. S. Engineer, in charge of the Brazos District, that there is a depth of 18 ft. of water at mean low tide at the mouth of the river, and that this depth extends not only from the Gulf through the jetties, but to 500 ft. above the lighthouse, which is some distance inland. The works at the mouth of the Brazos River were built in reference to a datum plane of average flood tides; this would make the depth at mean high tide over 19.8 ft. A more recent survey by the Government Engineers shows that there is, throughout the entire length of the jetty channel, a minimum depth of 20 ft. at mean low water, 21.8 ft. at mean high water, the datum plane of the works.

The feature to which attention is particularly called by the writer is, that although the high water came in July last, and the channel at that time was deepened to 18 ft. at mean low water, it has maintained itself through the seven months intervening between that date and the date of the survey by the United States Engineer.

A. F. WROTNOWSKI, M. Am. Soc. C. E. (by letter).—While the conditions at Aransas Pass, resulting from the construction of its disconnected reverse-curve breakwater, seem to have fulfilled in a measure the expectations of the Board of Engineers who planned it, much remains to be done before it can be accepted as a success in the improvement of tidal harbors. Mr. Wrotnowski.

The very essence of the jetty principle of concentration of the volume of water, whether it be tidal or fluvial, between parallel dykes, is at stake, in this so-to-speak experiment; and should it prove efficacious in all its features, the great reduction in cost alone would certainly recommend its adoption.

There is no doubt in the writer's mind that the curved trend of a breakwater or jetty adds materially to the forces for scour of the channel, over that of a straight jetty. The tendency of a moving body of water in a confined channel with an uneven and rough bottom will always cause eddies, and a crooked channel, more or less tending to cross and recross from side to side, making an irregular cross-section; but with a properly and regularly curved jetty the tendency of the volume of water is to "hug" the concave side of the channel.

The sectional area in a curve or bend of a stream, be it between jetties or in a natural water-way, is always greater than in reaches. This will always be the case in the whole length of the segment of the curve, and this applies equally to a channel in an open way, especially when it is guarded in its trend by a breakwater, as is the case at Aransas Pass.

The system of two tidal entrances or debouchures, as in this case, is especially applicable where tides are slight, as they are in the Gulf of Mexico, because, on account of the comparatively small tidal volume to be depended upon for scour, it is essential to secure all the volume possible to cause the required scour, and so this system may well be tried in such small tidal localities.

But at other points, for instance, on the Pacific, along the United States and Mexican coast, where the shores are very abrupt and defiant, and the tides rise much higher, it is doubtful if the system could be applied. For the port of Altata, State of Sinaloa, Mexico, the writer has had occasion to propose a single jetty for the maintenance of a given depth at the new entrance to this port, which was breached through the Peninsula about 10 miles south of Altata during the hurricane of November, 1896. The writer had then in contemplation a detached jetty, but after having made the surveys he found the conditions not adapted, principally on account of very unstable and deep shores. He found, also, that the volume passing in and out was sufficient to accomplish the required scour and keep a permanent depth; therefore, a single curved jetty was proposed. The Government, however, was not ready to expend the required funds for its construction until a better financial condition was apparent.

Mr. Wrotnowski.

The auxiliary tidal mouth in the Aransas Pass work is open to question as to whether it is an advantage. During high wave-energy, there will likely be brought into the channel quantities of sediment which will deposit to the lee, and which, during storms of long duration, will accumulate in such quantities as to cause much delay afterward in clearing the channel of such deposition, by the disturbed forces operating in the channel.

The writer has seen considerable such deposition through breaches, especially at Tampico and Vera Cruz; in one case, at the last-named place, where it filled a radial space of about 300 ft. to a depth of 20 ft., and it covered entirely the inside apron of the breakwater for over 500 ft. Not only did it deposit, but it kept on accumulating in the harbor for a time, causing excessive expense in dredging, until the breaches were closed. It is true, however, that in this case there was not sufficient tidal energy inside the harbor to carry away such deposit.

At Tampico the case was different, and the deposition which took place there was readily swept away, the conditions therefor being favorable.

Mr. Haupt.

LEWIS M. HAUPT, M. Am. Soc. C. E. (by letter).—Mr. Wisner cites an ideal case for the use of such a breakwater, but, unfortunately, it was in consequence of the effort to apply this ideal by "continuing the natural curve of the outlet" that caused the work of the Government to fail at Aransas, for the reason that the "natural curve" is the result of the external drift encroaching upon the advancing or convex side of the inlet, and is therefore on the far or wrong side of the channel. A jetty built on the concave side, therefore, deposits the littoral drift in and blocks up the channel, causing the bar to move more rapidly seaward. Hence, it is not generally good practice to continue the "natural curve" without an auxiliary structure abreast of it to arrest the drift, thus requiring two jetties if the far one is built first.

As several members appear to be under the impression that the single curved breakwater is not applicable to the mouths of sediment-bearing rivers, the writer desires to state his reasons for entertaining a different opinion. The objection is based upon the statement "that any diminution of velocity of flow produces deposition."

This is conceded as correct on general principles, but subject to the modifications as to effects of velocity in producing scour, as noted by the writer and confirmed by Professor Williams, and Mr. Wisner in his former papers. Hence, the only question arising is as to which form of construction produces the greater diminution of velocity, one curved jetty or two straight ones; and are their effects upon the outflowing currents at all similar?

The writer's position as to two rigid jetties which are practically parallel is, as already stated, that they act merely as an aqueduct to convey the effluent water and its load of sediment over the site of an

existing bar merely to be deposited in the sea beyond, where a new obstruction will be formed, unless there is a strong littoral current to prevent it. A single reaction jetty, on the contrary, produces a lateral movement of sediment, thus removing it from the concave side where the velocity is the greatest, because of the longer path, and notwithstanding the lesser slope, to the convex side, where it accumulates and forms a natural levee, which, in process of time, automatically adjusts itself to variations of the regimen of the stream. It thus happens that instead of all the material being carried or rolled to the mouth, as with two jetties, by far the greater portion of it is thrust aside before reaching the outer end of the concave jetty and is deposited outside of the navigable channel, and thus the growth of the bar seaward is greatly retarded. The result is that the one jetty not only costs far less to build, but is also much cheaper to maintain. Mr. Haupt.

These views appear to be sustained by the facts stated by Mr. Wisner, relative to the maintenance of the Eads Jetties at the South Pass of the Mississippi River, authorized in 1875, at an estimated cost of $5 225 000. These jetties were parallel and 1 000 ft. apart—too far, in fact, for the best results from the small portion of the discharge traversing that effluent—but being rigid, the readjustment could only be made by groins and inner jetties which contracted the channel to about 600 ft., thus increasing the cost, and at the same time producing alternations of velocity instead of uniformity, but resulting finally in securing the 30 ft. of depth desired.

Mr. Wisner, however, makes an important statement, viz.: "That the curvature of the channel was such that excessive depths developed near the concave jetty and caused sufficient deposit on the convex side to reduce the width at a depth of 26 ft. to less than 200 ft.," thus showing that the degree of curvature was excessive, since the reaction was too great for the contracted channel; also, that there was a resultant movement of material from the concave to the convex bank. The channel through the South Pass has several reversals of curvature and consequent "crossings," yet the depths are maintained. The east or concave jetty* was located as a transition curve starting from a tangent, with a gradually increasing deflection until it reached a radius of 11 720 ft.; its total length being 12 100 ft., nearly 2¼ miles. The west jetty was parallel to the other for nearly 3 000 ft., and thence to sea it curved with a radius of 15 000 ft.

Mr. Wisner states a fundamental principle when he says: "The amount of curvature given a channel fixes the width that can be maintained." Hence, the success or failure of the engineer depends upon a proper adjustment of his radii to the local conditions; if too short, the channel will be too deep and narrow, and *vice versa*. The happy medium which is best adapted to all stages must be determined.

* "The Mississippi Jetties," by E. L. Corthell, M. Am. Soc. C. E., p. 75.

Mr. Haupt. Hence, the writer is of the opinion that a single curved jetty, adjusted to the regimen of the stream, and having no restraining counter jetty to interfere with the natural deposition of material beyond the normal section of the channel, will not only cost less, but will give a far more satisfactory result as to maintenance.

In support of this view the writer submits a few brief extracts from well-known foreign authorities illustrating the action of curved channels. In his "Tidal Rivers" Mr. Wheeler says:

"A concave bank sets up a scouring action in the current by diverting the particles of the water from their straight course, causing that rotary motion and boring action which occurs in all bends, and which operates in deepening the channel along the concave side."

Mr. Stevenson says: "That it might safely be affirmed that a stream is most likely to follow a permanent course when directed by a concave bank."

Mr. Scott-Russell: "Where the curves were gentle, the natural bends should not be interfered with; that a river has an oscillating motion * * * the mere fact of the commencement of curvature would give it a tendency to continue that curvature, and the stream would go on oscillating regularly to the sea in curves of opposite curvature. Continuity of a system of oscillation should therefore be maintained."

Captain Culver says "that straight reaches are strictly to be avoided * * * since the deep-water track is acted upon by the most trifling causes, ranging from side to side at will * * * there is therefore no security whatever for the permanency of the deep water, either in a fixed channel or at the shipping berths."

The French Government Commission, in reporting on the improvement of the estuary of the Seine, advised that the training walls should be extended in a sinuous form, having a concave bend leading to the entrance to Honfleur Harbor, in order that deep water should be maintained at the entrance.

M. Fontain, in the rectification of the Rhine, avoided straight cuts and adopted curves.

From which it appears that the most experienced authorities recognize the value of curvature as a means of creating and preserving a channel in silt bearing, tidal streams and estuaries.

From all of which it would appear that for tidal estuaries or inlets on sandy coasts, where drift and wave bars obstruct the entrance, a single reaction breakwater should be placed to leeward of the proposed channel; while for sedimentary or delta rivers emptying into comparatively tideless seas, a single concave jetty, of long radius adjusted to the discharge, will best serve to create and maintain the best channel at least cost.

The cases are widely different, in consequence of the origin and direction of movements of the silt; the one being littoral and external, the other fluvial and internal; hence they require distinctively different treatments.

Vol. XLIII. JUNE, 1900.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

No. 870.

THE SOUTH TERMINAL STATION, BOSTON, MASS.

By GEORGE B. FRANCIS, M. Am. Soc. C. E.

PRESENTED AT THE ANNUAL MEETING, JANUARY 18TH, 1900.

WITH DISCUSSION.

INTRODUCTION.

The construction of the Southern Terminal Station in Boston by The Boston Terminal Company has reduced the number of terminal stations in that city to two: One, the Northern Station, serving the railroad lines entering Boston from the Northeast, North and Northwest, and taking the place of terminal stations formerly used by the Boston and Maine, Boston and Lowell, Fitchburg and Eastern Railroads; the other, the Southern Station, serving the railroad lines entering Boston from the West, Southwest and South, and taking the place of terminal stations formerly used by the Boston and Albany, Old Colony, Boston and Providence and New England Railroads; all the roads entering the Southern Station, except the Boston and Albany, being leased to the New York, New Haven and Hartford Railroad Company.

This latter station has just been completed, and its engineering features are described and illustrated in this paper. The project for

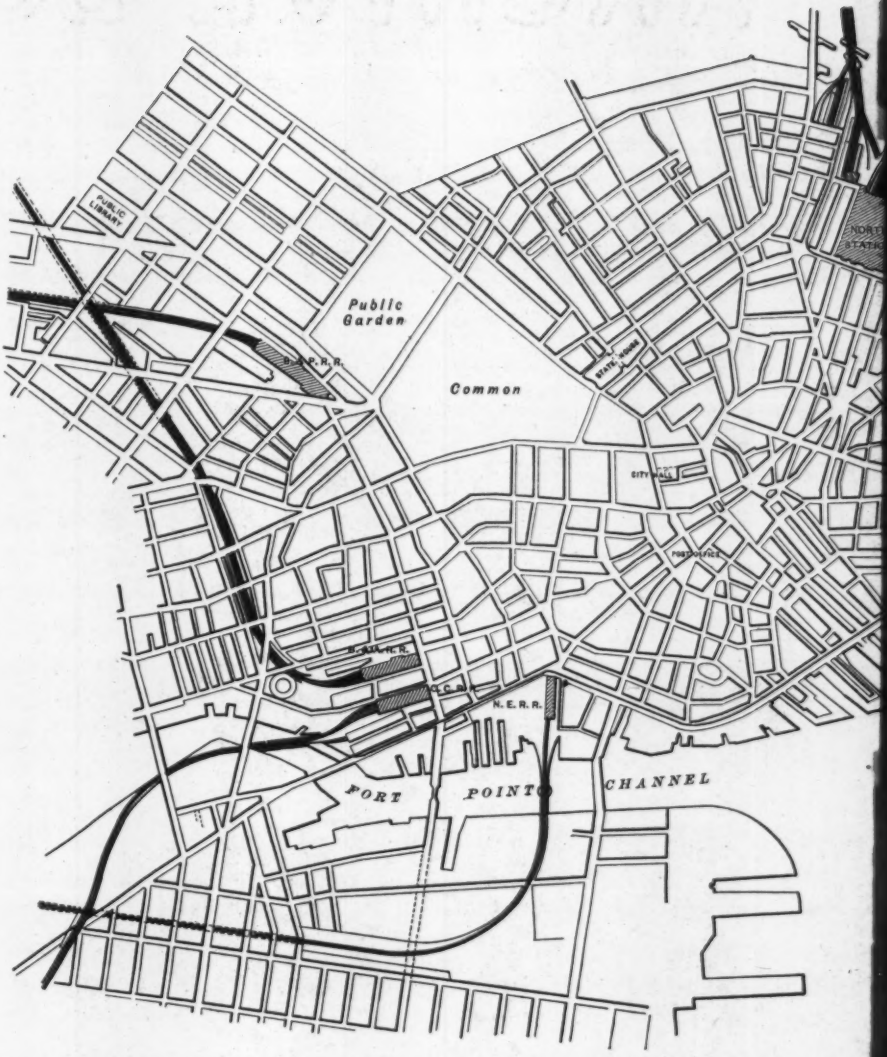
this station was conceived early in the year 1896; the charter was approved June 9th, 1896; and The Boston Terminal Company, consisting of the five railroad companies, as follows: The Boston and Albany Railroad Company, The New England Railroad Company, Boston and Providence Railroad Corporation, Old Colony Railroad Company, and The New York, New Haven and Hartford Railroad Company, each taking $100 000 of stock (all other funds being raised by terminal bonds), was immediately organized.

Surveys and plans were started July 1st, 1896; general plans were approved by the Mayor and Railroad Commissioners on December 22d, 1896, and January 4th, 1897, respectively; construction was begun in a preliminary way in January, 1897, and comprehensively in April, 1897.

The Old Colony and New England Railroads were transferred to the station January 1st, 1899; the Boston and Albany Railroad July 23d, 1899, and the Boston and Providence Railroad September 10th, 1899.

With a few minor exceptions the work at this date (September, 1899) is entirely completed. The site selected embraces the location of the old New England terminal at the foot of Summer Street, a part of the Old Colony terminal on Kneeland Street, as well as all the land lying between these stations and alongside of the latter, bordering upon the Fort Point Channel, a part of Boston Harbor. A portion of Federal Street, Mount Washington Avenue, and several minor streets have been abandoned, and Summer and Cove Streets and Dorchester Avenue have been extended and widened to serve public street traffic in their stead.

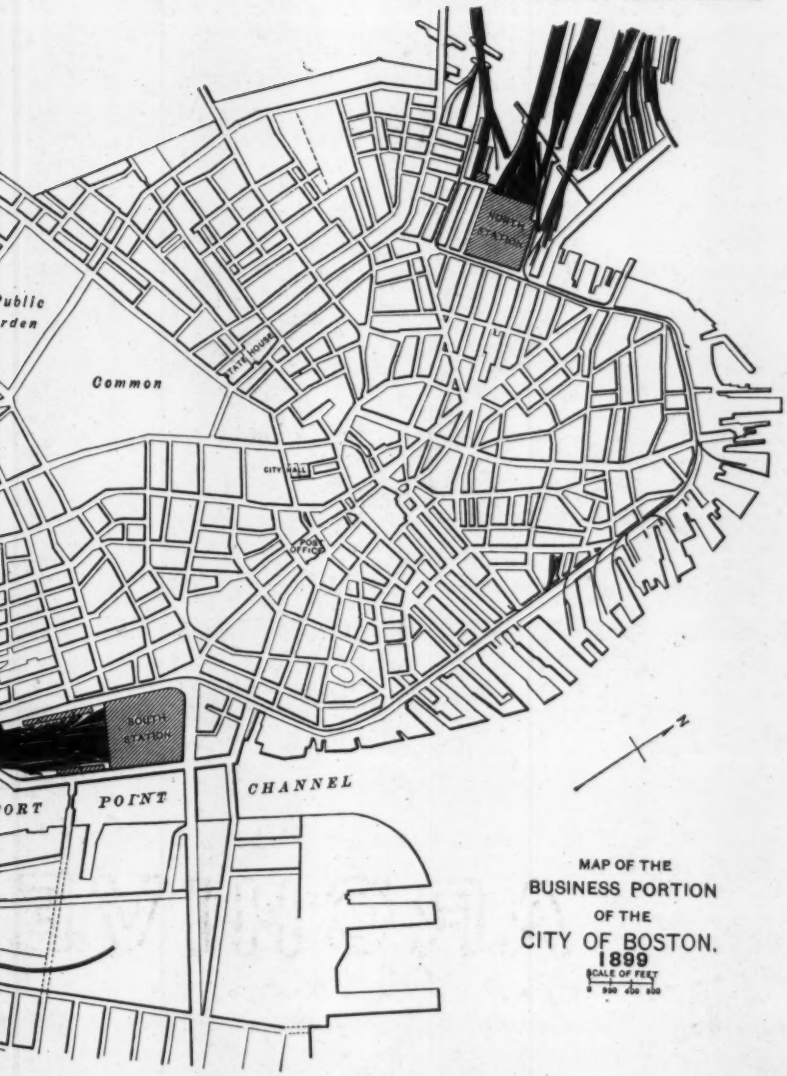
Aside from the terminal work undertaken by The Boston Terminal Company, first it has been necessary for the City of Boston to construct the new streets; land for the same, equivalent to that taken from streets, being given to the city by the Terminal Company, and to relay the public sewers and water pipes in the vicinity. This street work includes a sea wall along the Fort Point Channel about 2 000 ft. in length to sustain Dorchester Avenue; a new street and drawbridge 100 ft. wide for Summer Street extension; also, some bridge work in connection with Broadway and Albany Street. Since the plans for the station were approved by the Mayor and Railroad Commissioners, an additional street and bridge over the terminal grounds and Fort Point Channel, making an extension of Cove Street to Dorchester


PUBLIC LIBRARY
Public Garden
B. & P. R.R.
Common
STATE HOUSE
CITY HALL
POST OFFICE
B. & M. R.R.
N. E. R.R.
PORT
POINT
CHANNEL

MAP OF THE
BUSINESS PORTION
OF THE
CITY OF BOSTON
1896

0 500 1000 1500 2000
SCALE OF FEET

PLATE V.
TRANS. AM. SOC. CIV. ENGRS.
VOL. XLIII, No. 870.
FRANCIS ON BOSTON SOUTH TERMINAL.



MAP OF THE
BUSINESS PORTION
OF THE
CITY OF BOSTON.
1899
SCALE OF FEET
0 200 400 800

Avenue, has been proposed by the city, but the construction of the same has not yet been undertaken, and this street is therefore shown in dotted lines on the general plan, Plate VI.

Second, it has been necessary for the Old Colony, New England and Boston and Providence Railroads, through their lessor, the New York, New Haven and Hartford Railroad Company, to connect with the terminal location the first two by means of a new six-track rolling-lift drawbridge over Fort Point Channel, and the third by constructing a very difficult and expensive stretch of four-track road-bed about a mile long, from Dartmouth Street, on the Back Bay, so-called, to the terminal, alongside of the road-bed of the Boston and Albany Railroad, in the heart of Boston.

It is not intended in this paper to describe further the work done by the City of Boston or the work done on the connections by the New York, New Haven and Hartford Railroad Company, but rather to describe the work of The Boston Terminal Company exclusively.

The land taken (land, exclusive of streets), was owned by fifty-six private parties, three railroad companies and the Commonwealth of Massachusetts. A complete survey was made of the entire tract, and of each individual piece of land. These surveys have been filed in plat-book form for permanent record. Location maps and descriptions were also made for the public records, as required by the charter. The value of the lands thus taken for the terminal aggregated in the neighborhood of $9 000 000. It was necessary to make thorough search of land records in order to interpret the survey, as there were some joint dockage rights and wharfage grants which could not be otherwise understood.

Borings.

Borings were made at various points all over the site, and the results are faithfully set forth on the cross and longitudinal sections of the station. Generally, the site is underlaid with clay, mixed with some sand and gravel, and pile foundations were decided upon, the piles to be held firmly by friction.

Pile Test.

In order to get an idea as to the supporting power of piles in this clay, three ordinary spruce piles were driven, and loaded with 60 tons of pig-iron, with no resulting settlement. A description of this is

given with Fig. 1, Plate XII, more in detail. The value of such an isolated test is questionable, as the results might be totally different a short distance away. In this instance, however, the strata being quite uniform, the results were considered fairly indicative of what might be expected generally at this site.

Originally, the entire area was flooded by the tide at high water, and several series of wharves had been built, some of stone work, and some of timber cribs of all sorts and construction.

Encumbrances.

At the time of starting the work public travel existed over streets to be abandoned; sewers were in use upon which it was necessary to expend upwards of $100 000 to readjust them to the new conditions; water pipes and hydrants were in service; and telegraph, telephone, electric light, police and fire-alarm wires were in the way. Electric conduits and gas mains also encumbered the ground.

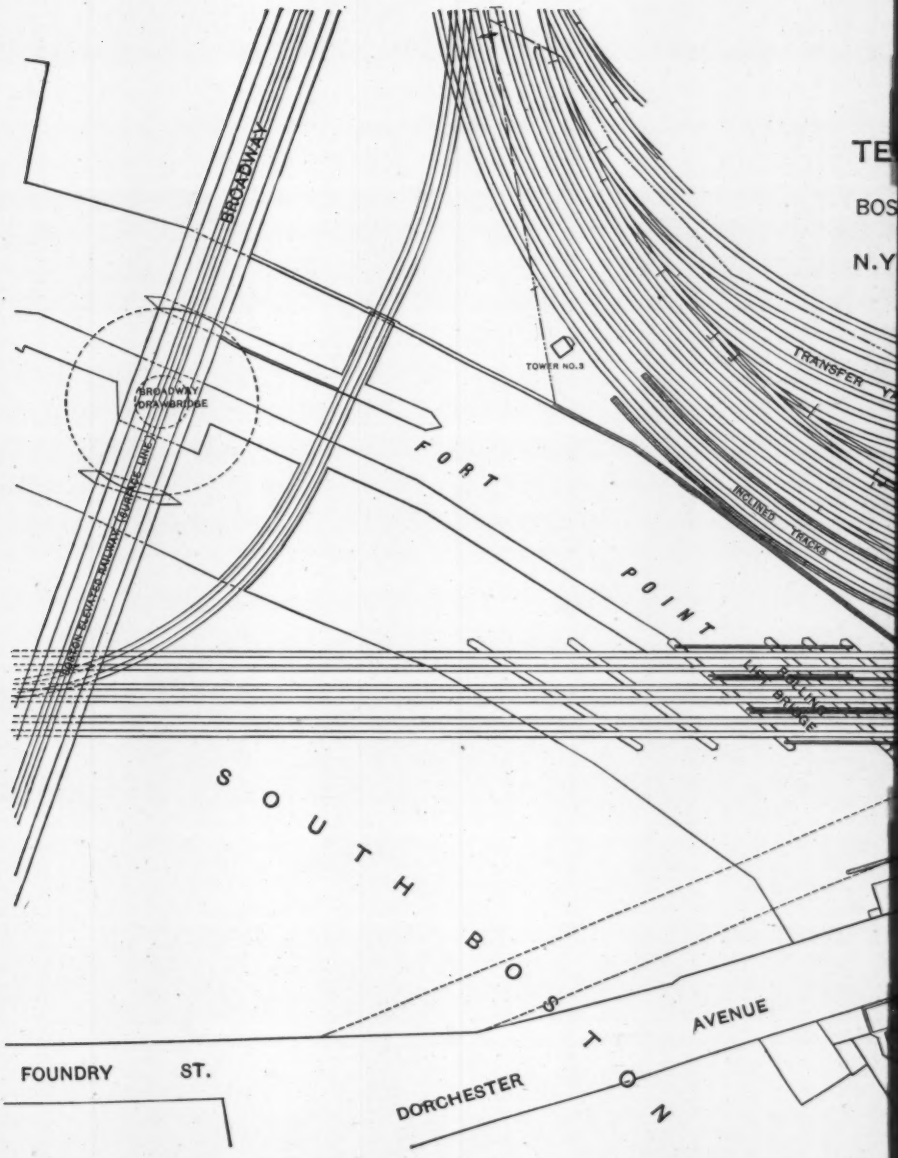
There were many leases of property not yet expired, even after the extinguishment of the owners' titles. All the coal in the great coal pockets, some of which were in the act of loading up from vessels, as well as all the stock and machinery in the various buildings, had to be removed before the buildings themselves could be torn down. About 210 buildings were removed, a few of them substantial modern structures, but mostly old dwellings, storage warehouses, freight houses, coal pockets, etc. There were a few manufacturing buildings and two very large gasometers.

General Plan.

While the above work was going on, many plans were made for the general layout of the entire station, finally resulting in the double-floor arrangement which has been constructed, the upper floor to be used for regular steam railroad trains, and the lower floor for suburban trains, using a special motive power, and running upon the same main road-bed.

As the railroad traffic about Boston has some characteristics different from almost any other American city, a short account of the line of reasoning which led up to this form of station will not be out of place.

Upon the railroad routes within 50 miles of Boston, about 50 000 000


BROADWAY
BROADWAY DRAWBRIDGE
BOSTON & ALBANY RAILROAD BRIDGE LINE
TOWER NO. 3
TRANSFER YA
INCLINED TRACKS
FORT POINT
L ST. BRIDGE
SOUTH BOSTON
FOUNDRY ST.
DORCHESTER AVENUE

GENERAL PLAN
OF
TERMINAL STATION
AT
BOSTON, MASSACHUSETTS,
FOR THE
N.Y.N.H.& H. AND B.&A.
RAILROADS.

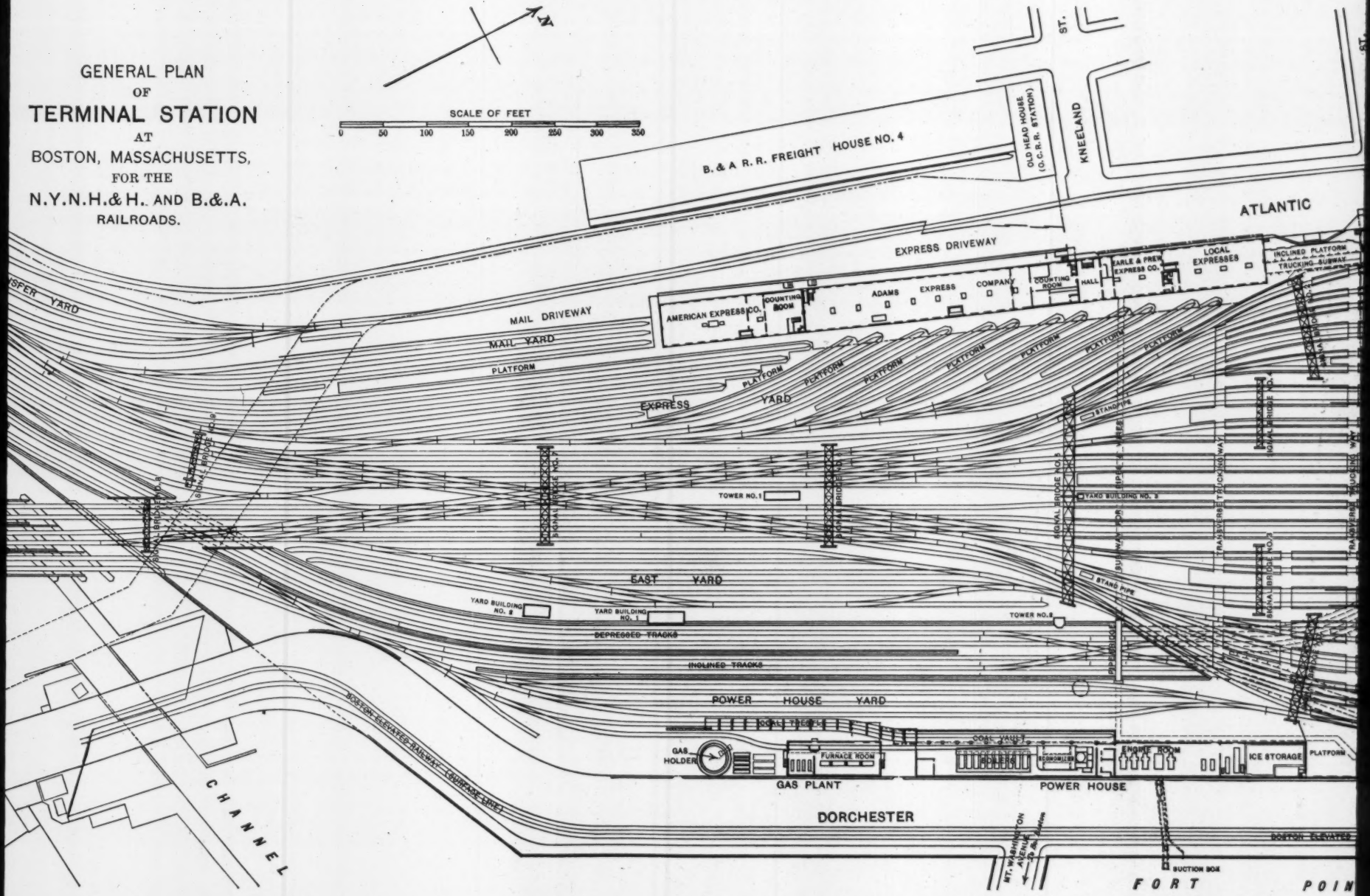
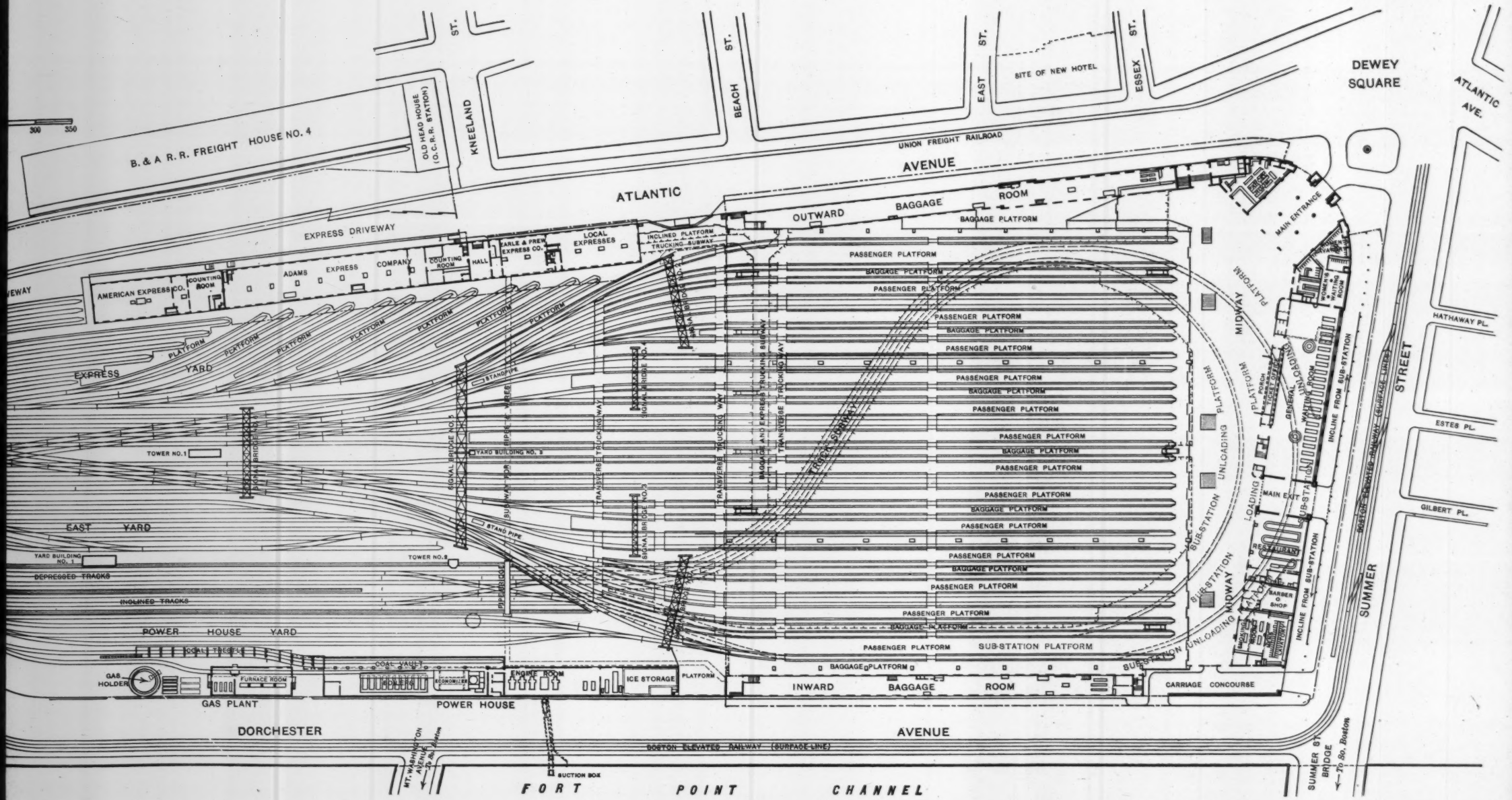


PLATE VI.
TRANS. AM. SOC. CIV. ENGRS.
VOL. XLIII, No. 870.
FRANCIS ON BOSTON SOUTH TERMINAL.



passengers are carried to and from the city each year, nearly equally divided between the North and South Stations.

The close approximate population within the 50-mile, or say, suburban, limit is 2 392 000. Within the same limit around Philadelphia it is 2 289 000; around Chicago, 1 188 000; around St. Louis, 874 000; and around New York, 4 754 000; New York, only, exceeding Boston. These figures show the reason for large terminal facilities in Boston, and the existence of the South Terminal Station.

It was decided at an early date that the new station must be so built that the new motive powers, electricity and compressed air, one or both, could be used therein; that a large increase in suburban trains, due to smaller train units and more frequent service, could be had, and that every effort possible should be made to remove the handling of baggage from the passenger platforms.

The first plans contemplated only a single floor for train service, but after arranging as well as possible for the various controlling features, making numerous studies for the exclusion of baggage trucks from the passenger platforms, and developing several ways of expeditiously handling electric cars, considering both high platforms on a level with the floor of such cars, and low platforms as an alternative, it was found that such unusual features tended to use up space, and that instead of 30 or more tracks, which it was hoped could be had, the number was reduced to 28. This was discouraging, as the old stations contained an aggregate of 25 tracks, and the increase of only 3 did not promise that opportunity for future growth which it was reasonable to expect.

There being no reasonable hope that a greater width of land could be secured, attention was directed to the possibility of divorcing the suburban, or short-distance, service, from the long-distance service, and placing the former at a different level, thus doubling the room for tracks on certain areas. The first sketch of this sort diverted the proposed suburban tracks about half a mile from the station, and by a gradually rising grade brought several tracks on either side of the train shed to a level about 24 ft. above the main floor. These were to be stub tracks, like those on the main floor.

This arrangement did not do away with the necessary switch system for making up trains outside of the station for each floor level; and upon its development, it was seen that if the elevated stub tracks could

be connected in the form of loops, the train movements would be very much lessened, and the number of switches at the entrance largely reduced. The extreme width of the station made the loop connection possible, and a plan was drawn accordingly. This was the first loop-track study. There was some doubt, however, as to the advisability of having upper platforms about 28 ft. above the main floor (which would be the case if high platforms were used) on account of the appearance of the structure, the difficulty of handling baggage, the nuisance from the smoke and steam of the locomotives below, as well as the advisability of avoiding stairs. The approaching grades were also steeper than was desirable.

The possibility of a suburban loop-track service upon a floor beneath the main floor was considered next. As the main floor of the station was designed at that time to be only about 6 ft. above the highest tides, it required to be raised to the highest level possible without the use of stairs at the entrances from the streets, which was prohibited. The main floor was raised 4 ft., and inclines were introduced at the entrances.

Even this placed the lower tracks, which were finally designed to be 17 ft. below the main-floor tracks, 7 ft. below very high tides. This immediately introduced the study of waterproofing this great area to shut out the water. Due consideration showed it feasible to do this without prohibitory cost. Placing the loop-tracks 17 ft. below the main floor and using high platforms for this kind of service made easier approach grades, stairways of 13 ft. rise instead of 28 ft., as was the case with elevated suburban tracks, simplified the handling of baggage, improved the appearance of the main floor, and did away with the smoke nuisance.

The main station building was also drawn, set back from Summer Street; so that inclined approaches could be had to or from the first loop-track, as well as to the main floor. Stairways were drawn from the main floor to the lower floor, so that all persons taking loop-trains at the station could reach them by going downstairs only.

Before the loop plan was adopted it was drawn in various ways and positions, sometimes with two loops, sometimes with four, sometimes with curved corners and some straight tracks; also tracks curved throughout. Studies were made of methods of handling the passengers and building the cars.

FIG. 1.—GENERAL VIEW OF WORK, JUNE 15TH, 1897.

FIG. 2.—GENERAL VIEW OF WORK, AUGUST 18TH, 1897.

The one great feature which made the loop system attractive was the making of this great terminal a through station for one kind of service and a terminal station for the other. Suburban trains do not carry mails, express matter, or baggage in any quantity. They carry people only, and should be run frequently and got out of the way rapidly. The through-station idea made it possible to handle as many trains at the terminal as it is possible to run over a main line, the trains leaving their passengers and moving on without switching. If the coming motive power should be electricity, through a third rail, it will not conflict with the immense switching system which must be maintained at so large a terminal.

As this station must eventually provide for suburban trains from the four main lines, such trains must leave the station at shorter intervals than trains could be run on either main line alone. This was provided for by arranging, with a minimum amount of switches, two loop tracks to be used alternately, each capable of holding several trains, one track to be filled while the other is being cleared. By this means trains can remain in the suburban station much longer than the actual time interval between trains. These loops could have been arranged to operate with even a less number of switches than designed, but it would have obliged all trains from some routes to use one loop exclusively, whereas it was desirable to make the forward loop available for all routes to its full capacity.

Taking up the various engineering features connected with the problem, a description will be given of each of the more prominent ones in about the order in which they confronted the engineer.

Coffer Dam.

The coffer dam which it was necessary to construct, and without which it would have been impossible to execute the work, merits a brief description.

As the borings show an underlying stratum of clay within the reach of a single length of timber, it was evident that a continuous timber wall driven into this clay close enough to prevent leakage in the joints would positively exclude the water. With this sort of a wall surrounding the work no trouble would result in execution. In some places this line of timber wall, consisting of 6-in. splined hard pine planks, in about 40-ft. lengths, was doubled to give sufficient lateral

strength to resist pressure from open water; at other places it was a single line.

Wherever this sheet piling was driven it was necessary, first, to remove entirely any stone or timber in the way. Any failure to do this was disastrous. There were some exasperating delays in the removal of the obstructions, obliging the use of several divers and a wrecking outfit, but eventually the entire terminal excavation was shut off from the salt water on three sides, and in some pockets on four sides, with this timber wall.

From the very start up to the present date there has been no flooding of the lower floor, the coffer dam having effectually done its work. The cost of this dam has been about $75 000. It is also of use in connection with the permanent shutting out of high tides, thus reducing excessive pressure on the waterproofing. By some it has been considered that the coffer dam presented the most difficult feature of the undertaking, and that without success in this particular, it would have been impossible to execute the work. The total length of this dam was about 3 000 ft.

Waterproofing.

The permanent waterproofing sheet, which underlies the whole lower floor, consists of ten layers of tarred paper, swabbed together with hot coal-tar pitch. The layers are carefully lapped, and all built in place. This continuous sheet of waterproofing, amounting to 56 000 sq. yds., or upwards of 10 acres, is laid, where horizontal, upon a concrete base, troweled to a smooth surface. In some of the softer places boards were laid down under the concrete base, but usually this base, 6 ins. thick, rests upon the ground. Where the waterproofing is on the vertical walls, it is backed up with 8 ins. of brick work. In all cases the horizontal sheet lies between the pile heads and the granite masonry above.

Upon the waterproofing sheet, where it is not weighted down with walls, there is placed a loading of a cheap grade of Portland-cement concrete, sufficient to resist the upward pressure of the water, which is, on the horizontal layers, about 500 lbs. per square foot, this pressure being dependent, of course, on the elevation of the sheet, with reference to the ground-water level. In some places the weight of the walls is far in excess of the water pressure; in others, where possible, the pressure is resisted by inverted concrete arches.

FIG 1.—GENERAL VIEW OF WORK, OCTOBER 26TH, 1897.

FIG. 2.—GENERAL VIEW OF WORK, DECEMBER 17TH, 1897.

A drain-pipe channel underlies each loop track for its entire length, and these discharge into the sump well near the power-house, where both rain water and any leakage through the waterproofing finds its way, either through the automatic tide gates at low tide, or is pumped at other stages of the tide into the Fort Point Channel. The tarred paper is introduced into the waterproofing sheet to give it a fiber, and the pitch to make it watertight. It was not desirable to have the sheets of paper of great thickness, as that would permit the paper to split in its own thickness, but rather a thin paper, which would saturate to a greater extent, and to make up for thickness by a greater number of sheets, the total number being a matter of judgment. The coal-tar pitch should spread under pressure, and when warmed in the hand should spin out as fine as a hair. This combination of pitch and paper was selected in preference to a layer of asphalt, as it could be bent, twisted, knuckled, abused and handled more readily than the asphalt, which, if disturbed very much, it was thought, would crack clear through. It is not believed that the pitch will volatilize in its secure position away from air. Up to the present time no leakage in this great area has been discovered.

The waterproofing work was carried on regardless of the temperature. An 8-in. brick wall was found to be the most suitable backing where the sheet is brought up back of the walls. Care had to be taken that the walls should not slide sideways upon the pitch before the concrete bracing in front was in place.

To determine the pressure likely to be exerted on the under side of the waterproofing sheet, the following line of reasoning was used:

	Elevation.
Mean high-water level is about	10.00
Extreme high-water level (during a severe storm in 1851) is registered as	15.74
Mean low-water level is about	0.00
Extreme low-water level is about	—3.00
Ground-water level some distance back from the water front, by observation, is about	9.00

showing that high tides do not remain up long enough to penetrate any great distance through the ordinary earth filling.

As the sheet-pile cofferdam, with the top at elevation 16.00, put in to enable the work to be carried out, was to be left in the ground,

it was reasoned that extreme high tides would not penetrate beyond the dam, and therefore that an assumed elevation of 12.00 would be safe for any possible condition. Had the coffer dam entirely surround the site, it is quite likely that water would have risen inside of the dam to the top of the sheathing, but as the dam does not extend entirely around the work there is ample opportunity for the ground-water to stand at its usual level.

Having assumed the water level at elevation 12.00, as giving the maximum hydrostatic pressure, it only remained to calculate the required loading of concrete for any given level of the waterproofing sheet. To distribute further any load from columns over the sheet as as far as possible, old steel rails were introduced into the concrete wherever it was possible. These are shown in the various sections. In the case of the light loading under the loop-tracks they were used to distribute a surplus loading on either side to the area under the tracks. They are also placed under the retaining walls so as to project as far as possible.

Piling.

All the piles used have been of spruce, varying from 25 to 40 ft. in length, with a minimum of 12-in. butts and 6-in. tips. The specifications called for these to be driven their full length into the ground, or until the penetration should not exceed 12 ins. in twelve blows from a 2 000-lb. hammer falling 20 ft., with the hammer line attached, or its equivalent. The spacing generally has been 2 ft. 6 ins. center to center, and in some places a little less. The loading has ranged between 8 and 10 tons per pile, which is reasonable in friction ground, and with piles of the above character. In some places the penetration was from 3 to 8 ins. at the last blow, and where this occurred, additional piles were used to reduce the loading. In all, about 43 000 piles have been used. The cost has varied from $3.10 to $5.00 each, in place, depending upon the length.

Concrete.

Three different classes of concrete have been used in the work.

First.—Ordinary natural-cement concrete, one part cement, two parts sand and five parts screened gravel.

(In all cases on this work screened gravel was specified in order to avoid sharp stones against the waterproofing sheet. Experience,

FIG. 1.—GENERAL VIEW OF WORK, FEBRUARY 10TH, 1898.

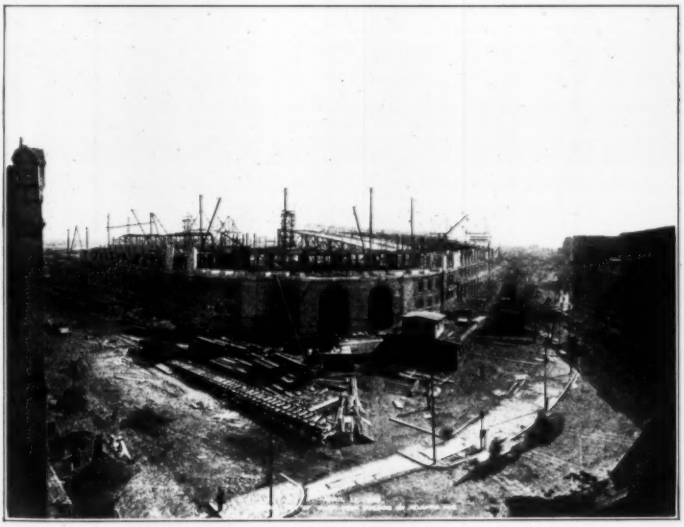


FIG. 2.—GENERAL VIEW OF WORK, APRIL 11TH, 1898.

however, has shown that this was not absolutely necessary. This has been used for backing masonry in granite-faced walls, and was specified on account of the necessity of having a smoother surface on the rear of the walls against which to build the water-proofing sheet. It was discovered, however, by the contractor's men, that they could lay up a very smooth face out of rubble granite and mortar when required for the waterproofing sheet, and at about the same cost. This was allowed in places, as it made a slightly heavier and better wall when well done. Had the smooth-backed rubble wall been specified, it doubtless would have raised the price of rubble over concrete, on account of the prospect of the contractor having to build a double-faced wall, but when the trial was made it was found that the difference in cost was insignificant, as the workmen soon adapted the work to the requirements.

Second.—Portland-cement concrete, one part cement, two parts sand and five parts screened gravel.

This has been used where it became a part of a foundation to support columns or piers, where it was the foundation support for the waterproofing sheet, where used as an inverted arch, and in places where it was necessary to deposit concrete in water.

Third.—Portland-cement concrete, one part cement, three or four parts sand, and six or seven parts screened gravel, the proportions depending on the size of the individual pieces of gravel.

This was used as a loading for the waterproofing sheet, and was meant to be only a reasonably stiff loading, but it has proved to be almost as hard as rock, and cannot be cut or broken except with stone-cutting tools. In a few instances concrete of this class was used as a backing for walls, rather than cause delay by waiting for the usual material. When used as a waterproofing loading, it was not rammed as hard as otherwise, as it was desirable that seepage water should find a way to drains provided, without coming to the surface. Nearly all the concrete used on this work was mixed in a Cockburn Barrow & Machine Company concrete mixer. The work was carried on regardless of temperature, and in cold weather no special provision was made except to thoroughly heat the water, sand and gravel before mixing, which was done by steam pipes in large bodies of the material. The cost of the concrete has varied from $3.30 to $5.00 per cubic yard, according to classification and contract.

Cement.

The Portland cement used throughout the work was the Alpha brand of American Portland cement. The masonry and concrete in the work required a large quantity of cement, but did not demand that an extra fine quality be used, as might be the case with work done under running water. Specifications were purposely drawn to admit the use of any good average brand, foreign or American, either slow or quick-setting, with the object of opening the bidding to lively competition and a low rate. The specifications required, for Portland cement, a tensile strength of 200 lbs. per square inch in 24 hours, having stood in air until set, and the remaining time in water; and for natural cement, a tensile strength of 100 lbs. per square inch when tested in the same manner.

This method secured the cement at very low rates. About 75 000 bbls. of Portland cement and 20 000 bbls. of natural cement have been used, and, in the case of the Portland, it is believed that nearly $1 per barrel was saved over what would have been the price if a rigid specification had been adopted and a limited number of brands allowed.

A great many tests for tensile strength were made, and in no case was it necessary to reject cement. Owing to the large consumption of the mill product, only a small amount of cement could be kept on hand, and it was impracticable to make more elaborate tests before the cement was used in the work.

The proof that the cement is of a most excellent quality lies in the actual character of the concrete after it was put in place, even the poorest quality of concrete, consisting of ten parts of sand and screened gravel to one part of cement, requiring the use of hand stone-cutting tools where it has been necessary to open it for pipe ducts or other purposes.

Owing to the reduction in the cost of manufacture, and the ability to use Portland-cement mortar in zero weather without injury, and also the possibility of obtaining excellent mortar with a very small proportion of cement, making the cost of work done on a par with natural cements, the tendency to use Portland cement exclusively will undoubtedly increase very rapidly.

FIG. 1.—GENERAL VIEW OF WORK, JUNE 20TH, 1898.

FIG. 2.—GENERAL VIEW OF WORK, AUGUST 24TH, 1898.

Stone.

Of course, in eastern New England, granite is about the only stone which can be obtained in large quantities for building purposes, and granite was used throughout in all foundations and retaining walls. For the face of the walls of the subway and the train-shed piers, the specifications required that they should be laid in courses varying in height from 15 to 26 ins., alternating header and stretcher work, headers never narrower than their rise and 5 ft. long, if the wall would permit; stretchers limited to 8 ft. long, beds to be as wide as the rise, to be without slack spots of greater depth than 1 in. All joints to be not more than 1½ ins. for a distance of 12 ins. back from the face. On account of having a granite face and generally a concrete backing, two prices were used, one for face and one for backing. The face masonry has generally cost a little less than $10 per cubic yard.

The granite used in the work was brought from the following quarries: Stony Creek, Conn.; Milford, Mass.; Rockport, Cape Ann, Mass.; Pigeon Cove, Cape Ann, Mass.; Mt. Desert, Me.; Croacher's Island, Me.; Deer Island, Me.; Northbridge, Mass.; Fitchburg, Mass.; Pascoag, R. I., and Milford, N. H. In addition to this, large quantities of granite from Quincy, Mass., found in the old work, were used again.

Construction Drawings for Piling, Masonry and Tracks.

A scale of one-eighth of an inch to a foot was adopted for pile, masonry and track plans. This divided the work into a number of sectional sheets about 4 ft. wide and 8 ft. long. The longitudinal direction of each sheet took in the entire width of the property. The architects' drawings were also made on the same scale.

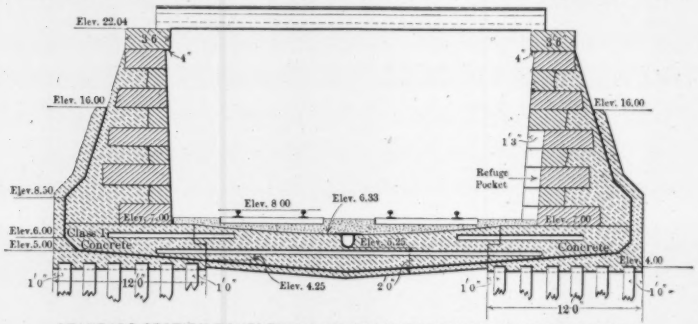
A main base line was adopted and stationed on the longitudinal axis of the train shed, and cross-sectional base lines at intervals of 100 ft. were set out on the ground. All primary locations were made as so much right or left from stations on the main base line.

The first series of plans covered the pile-driving. Each pile was shown, and upon an office blue print the result of each day's work was marked off, and a summary tacked on to the progress sheet.

The second series of plans covered the foundation masonry. Sufficient sectional plans were made on these sheets to illustrate the details, and they were complete in every respect, serving to govern the execution and laying out of the work.

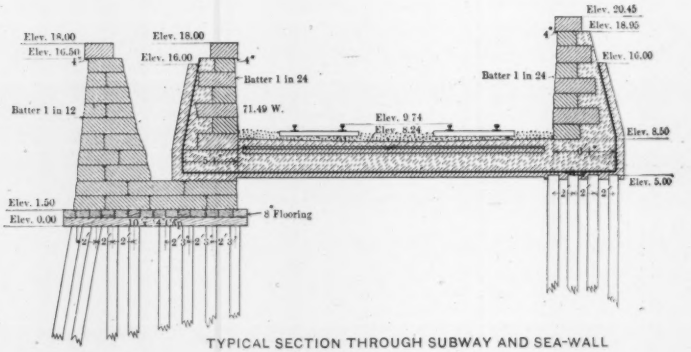
A third series covered the track work, and showed the station of each switch point, frog point, point of curve, apex, grades, each switch tie, interlocking supports, signal posts and bridge supports, and, in fact, everything necessary to make a quick field layout of the ties and tracks.

These plans, being all on the same scale, could be laid over one another, and any important discrepancy be detected instantly. No



STANDARD SECTION DOUBLE TRACK SUBWAY THROUGH "PLACE OF REFUGE"

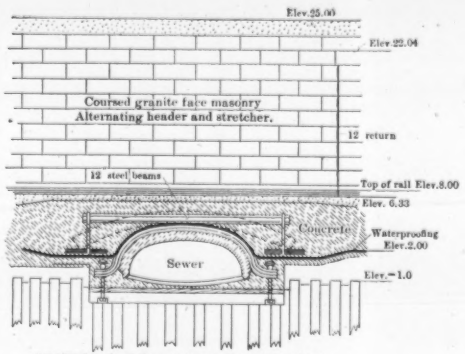
FIG. 1.



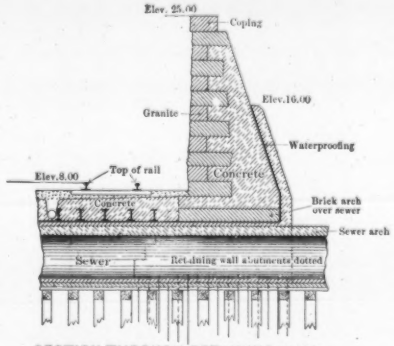
TYPICAL SECTION THROUGH SUBWAY AND SEA-WALL

FIG. 2.

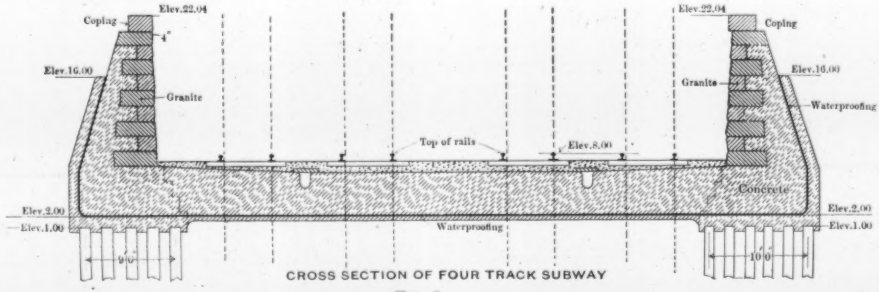
errors of any moment whatever occurred in the layout of the work, and it was, no doubt, due to the comprehensive manner in which these plans were prepared that such a clear understanding could be had by each workman, and that rapid, sure and economical progress could be made. These plans are so extensive that it is practically impossible to reproduce them for this paper, and only fragments have been introduced.



SECTION THROUGH SEWER IN SUBWAY AND ELEVATION OF WEST WALL

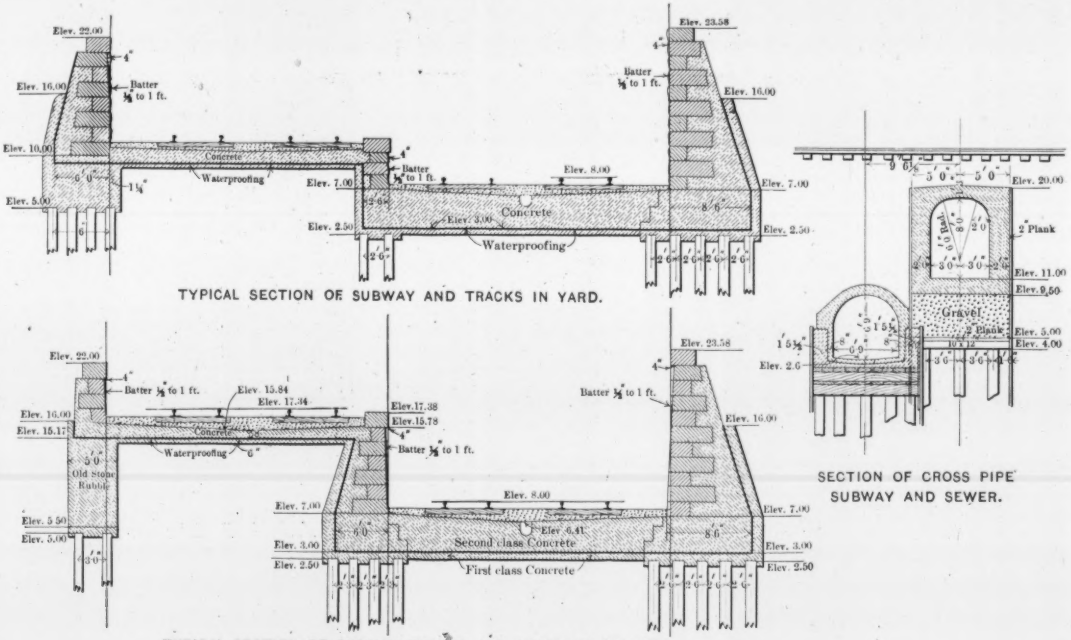


SECTION THROUGH RETAINING WALL AT SEWER CROSSING



CROSS SECTION OF FOUR TRACK SUBWAY

FIG. 3.



TYPICAL SECTION OF SUBWAY AND TRACKS IN YARD.

SECTION OF CROSS PIPE SUBWAY AND SEWER.

TYPICAL SECTION OF SUBWAY TRACKS AND SUPPORTS IN YARD.

FIG. 4.

FIG. 1.—GENERAL VIEW OF WORK, OCTOBER 24TH, 1898.

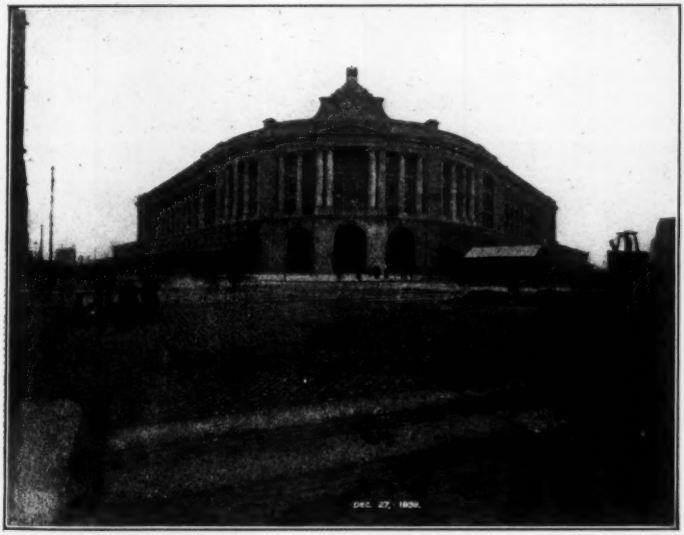


FIG. 2.—GENERAL VIEW OF WORK, DECEMBER 27TH, 1898.

Track Arrangement.

Not the least of the station problems most vital to the successful operation of the station is the track arrangement necessary just outside of the station for the safe and expeditious handling of the trains. It is believed that the arrangement adopted is simple and ample. By it the station may be operated as a unit, incoming trains on one side

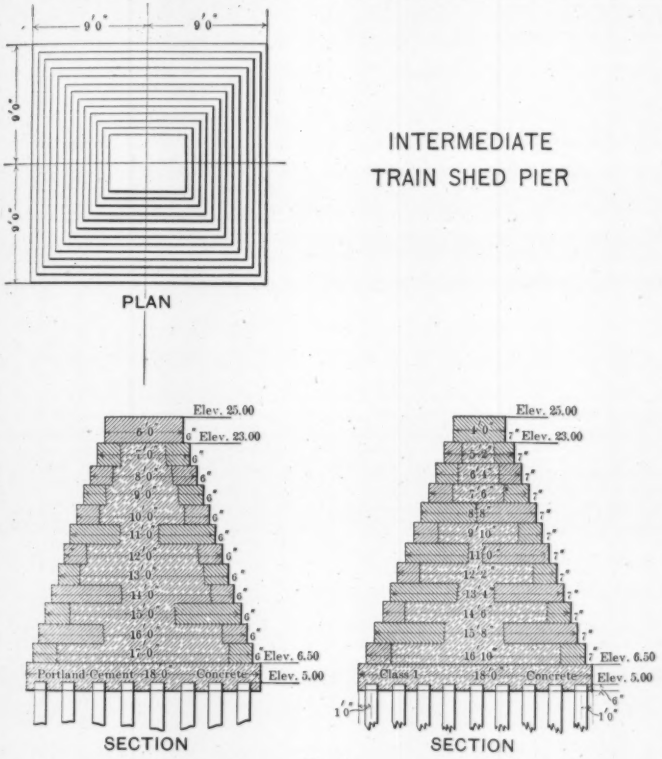


Fig. 5.

and outgoing trains on the other side, or in four sections, one for each of the four main lines. There are eight parallel main tracks, two for each of the main lines, through the throat of the yard; and, to enable trains to pass from either side of the throat to the other side there are two parallel tracks crossing the mains from either side in the shape of an X, or, as it is called, a double-track scissors-crossing, through the

eight main-yard tracks. Another single-track scissors-crossing, through the four main tracks, is provided for the subway or loop-track system.

The distance between the head house site and the Fort Point Channel, also the curved tracks connecting two of the main lines, was not great enough to permit of as easy curves for the development of the eight main tracks into the twenty-eight train-shed tracks as was desirable, but these curves have been kept down to a minimum radius corresponding to 10° curves.

The express-yard tracks have been arranged to give the greatest length of track opposite platforms, and, at the same time, to permit of shifting cars with as little disturbance of other cars as possible. Other yard-tracks are arranged to suit the various desires in yard work.

Number eight frogs have been used for the main-track crossings and for nearly all the regular turnouts on the main yard lines.

Number six frogs have been used for the subway crossing, and in certain places in the yards.

The express buildings were placed upon the westerly side of the yard, and the power-plant buildings upon the easterly side, this arrangement seeming best for each service.

Platforms.

With the exception of the two exterior tracks on each side, the tracks are spaced 23 ft., center to center, making the passenger platforms 14 ft. wide. There are two passenger platforms, one on each side of the shed, 23 ft. wide, with track spacings of 32 ft., center to center.

The baggage platforms are 8 ft. wide, track spacing 17 ft., center to center. The arrangement for baggage handling is such that there are nine baggage and express trucking platforms the entire length of the train shed, independent of the passenger platforms, and an underground passage to allow the baggage trucks to cross beneath the tracks. This method, which it is believed will very much reduce the baggage nuisance generally experienced, required seventeen baggage and express lifts, eight in the train house, four in the baggage rooms, and five in the express buildings. Those in the baggage platforms are protected with iron fences and gates with automatic attachments. These lifts are described, together with the other electrical and mechanical features of the power plant, in another part of this paper.

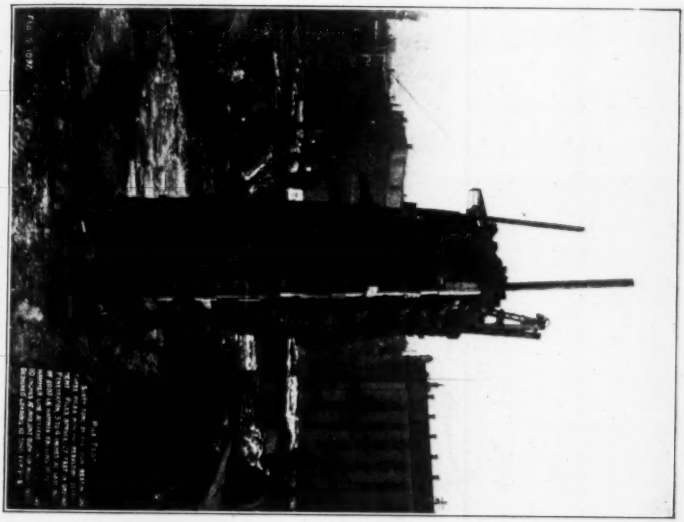


FIG. 1.—PILE TEST.

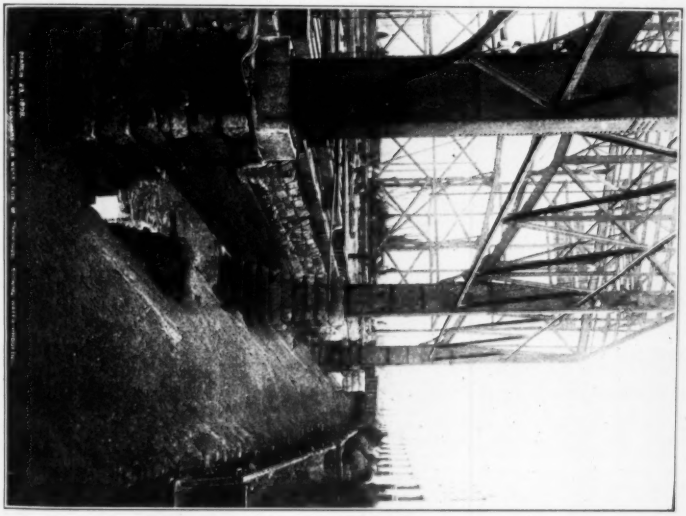


FIG. 2.—PIERS AND COLUMNS ON WEST SIDE OF TRAIN SHED.

Some of these platforms extend about 300 ft. beyond the end of the train shed, making them 900 ft. long. They were built of timber (hard pine), surfaced, 2 x 4 ins., and tongued and grooved, for a first construction, on account of the tendency of the newly filled material to settle irregularly over the old piers and docks. Eventually it is intended that they shall be granolithic. They are so constructed that, by removing cleated sections screwed down over the sills, they can be tamped up, similar to track, from time to time.

Rail.

The track rail used throughout the terminal is the New York, New Haven and Hartford Railroad standard, 100-lb. rail, $5\frac{1}{2}$ ins. wide and 6 ins. high, with rail head $2\frac{3}{4}$ ins. wide.

Frogs and Switches.

All regular turnout switches have points 15 ft. long and five bridle rods, connected to clips with turned bolts, nuts and cotter pins. Clips are connected to webs of rails by two bolts.

Slip switches have points 15 ft. long for the number eights and 15 ft. long for the number sixes, with four bridle rods, fastened as indicated above.

All slip crossings have a $\frac{3}{8}$-in. x 7-in. steel plate on each tie from end to end, and one tie beyond.

All slip crossings have movable frog points (with two bridle rods), 10 ft. 10 ins. long for the number eights and 8 ft. 4 ins. long for the number sixes. All other frogs are rigid bolted frogs.

Guard rails are 10 ft. long.

In the diamond crossings, the rails are double for practically the entire diamond. All switches and frogs were made by the Ramapo Iron Works.

Bumping Posts.

There are 54 Ellis patent bumping posts, all equipped with the standard 100-lb. rail.

Ballast.

All the ballast in the subway and its approaches is a broken trap-rock ballast from Meriden, Conn. All other ballast is ordinary gravel ballast.

Ties.

All regular 8-ft. ties are of chestnut timber, 6 x 8 ins., and all special ties are Georgia hard pine, sawed to dimensions, generally of 7 ins. depth by 9 ins. face, except some ties with 10 ins. depth for special places.

No tie preservative has been used. Upon all curves the Goldie tie-plates have been placed, and, wherever necessary, substantial cast-iron rail braces have been used.

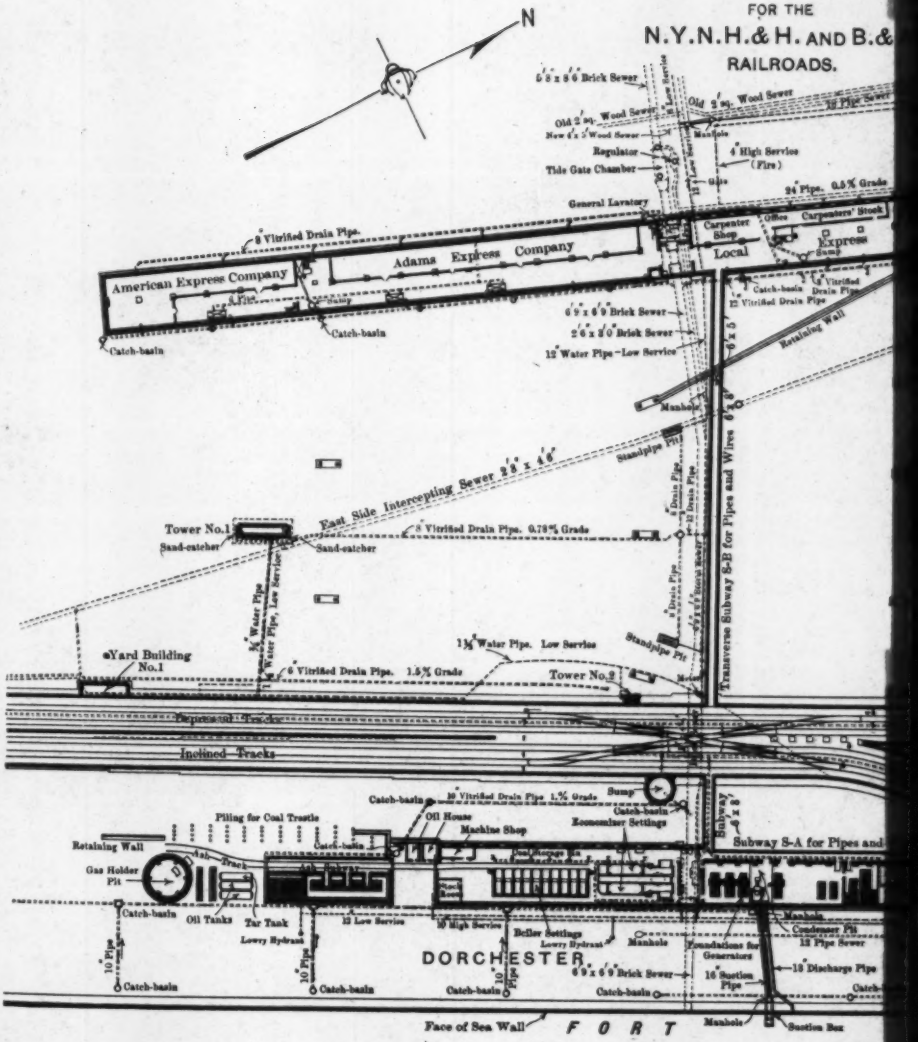
Steel Flooring.

The "Thomson" shape, or rectangular trough floors were adopted in preference to any type of floor with flaring sides, on account of the necessity of spanning the subway tunnel with the thinnest possible floor over all. Girders could not be arranged so as to come up between the ties, due to the various skew positions of the subway, therefore a floor of shapes with small depth could not be made on account of the span, which was 28 ft. in the clear. The troughs are made in about 18-in. depths, and, in order to get sufficient flange metal, cover-plates are used. The design is such that if at any time in the future the position of the tracks is altered, no alteration will be required in the floor. Impact allowance was disregarded in the floors in the train shed, as the speed of trains there cannot be great. Having adopted this type of floor on the greater portion of the area, it was thought best to continue its use over the remaining portion. The weight of the floor girders, columns and troughs complete is 94 lbs. per square foot. Ventilation from the subway tunnel is arranged for through the ends of the inverted troughs, and cast-iron grating ventilators in the baggage platforms over the lines of the retaining walls.

The entire top of the steel troughs and girders, after being painted with two coats of red lead and oil paint, are swabbed with an asphaltic swabbing material, costing about 15 cents per horizontal square yard of floor, according to the following specification:

"Swab the entire upper surface, vertical and horizontal, of all the steel trough floors with one coat of mineral pitch and oil put on at a temperature of about 350° Fahr., and mixed in a proportion of about 85% of mineral pitch (commonly called 'Trinidad Asphalt') and 15% of crude black oil (residuum of petroleum), if applied in warm weather,

BASEMENT PLAN
OF
TERMINAL STATIO
AT
BOSTON, MASS.
FOR THE
N.Y.N.H.& H. AND B.&
RAILROADS.



ENT PLAN
OF
AL STATION
AT
ON, MASS.
OR THE
& H. AND B. & A.
LROADS.

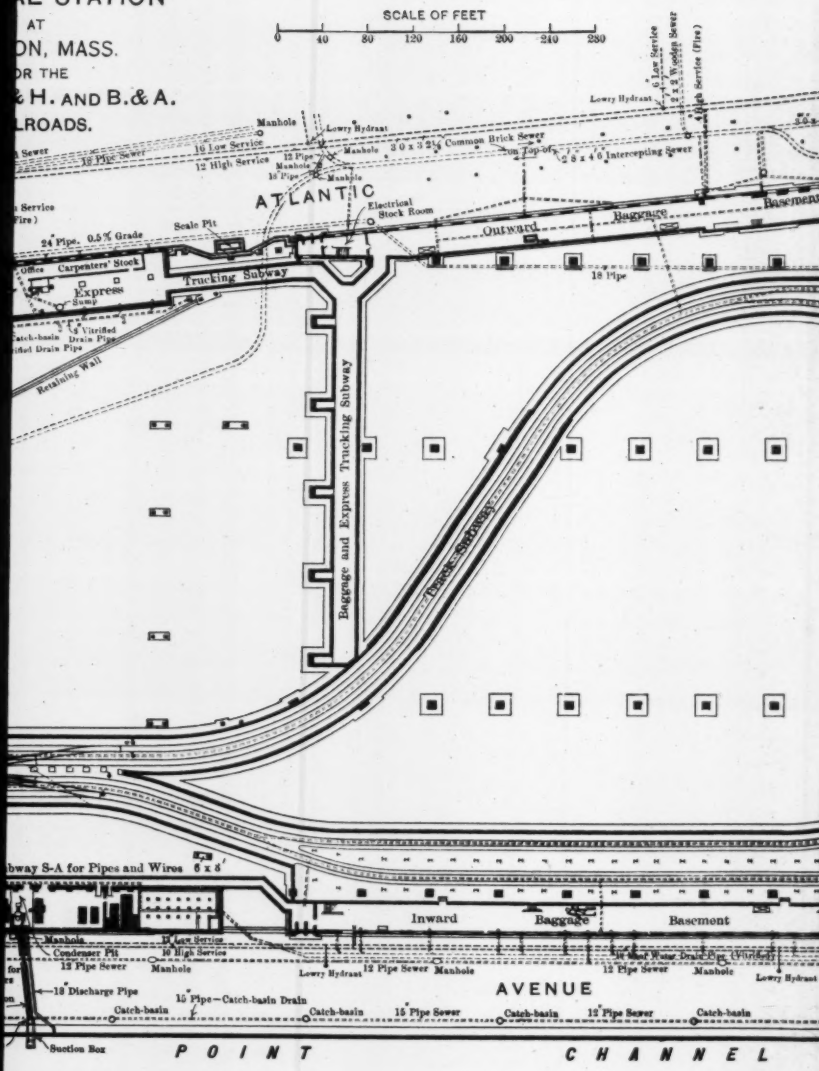
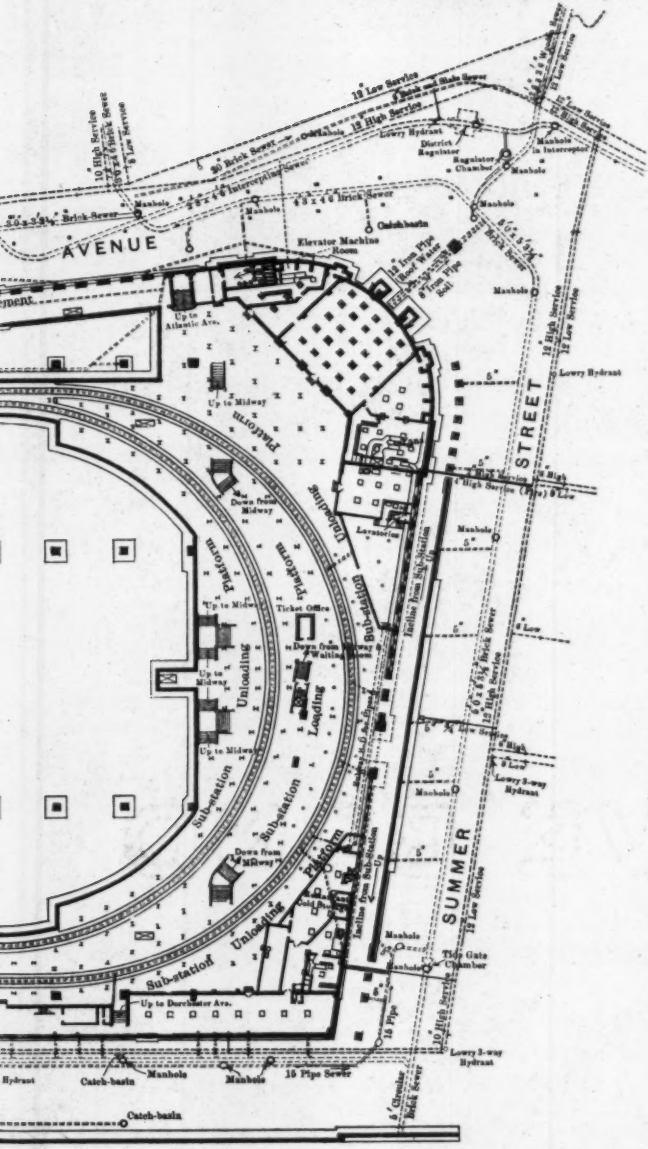


PLATE XIII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. XLIII, No. 870.
FRANCIS ON BOSTON SOUTH TERMINAL.



or of about 65% of mineral pitch and 35% of crude black oil if applied in cold weather, varying the proportion within these limits to suit the temperature. In warm weather the swabbing on vertical and inclined surfaces to be done only a small amount in advance of the ballast work, on account of the tendency of the mixture to crawl, under a hot sun. The mineral pitch to be either the trade article known as 'Refined Trinidad Lake Asphalt,' which contains no oil, or the article known as 'paving cement,' which is the same with an addition of about 15% of oil, and which oil shall be considered a part of the above-named oil proportion or percentage. The swabbing mixture to be prepared and heated in kettles at the site of the work. All troughs and iron to be swept clean before being treated, and no work to be done in the open air in wet weather. No gas-tar or other preparation to be used. Where steel flooring joins girders, or at other open joints, an asphaltic mastic shall be used to close the openings in a thorough manner."

Upon this swabbing the gravel ballast is deposited. The design of this swabbing is to coat the steel with a substance oily in its nature, many times thicker than any paint, and to thereby keep the water away from actual contact with the steel as long as possible. It was not advisable to place this swabbing coat where it could escape through the open connections, as it has a tendency to run in hot weather, causing an unpleasant appearance where it shows below.

Midway Floor.

The space between the head house and the train shed proper, called The Midway, is directly over the suburban station, and this area is supported on steel columns. Brick arches, with enameled brick lining, are turned from beam to beam, as a suitable ceiling for the suburban station. Upon the top of the arches, the floor is leveled up to the underside of the asphalt surface with a concrete made up of one part Portland cement, five parts fine cinders and ten parts coarse cinders. The asphalt surface is the usual Barber asphalt pavement, made of Trinidad Lake asphalt. This was laid 2 ins. thick, at a temperature of about 250° Fahr., and was rolled to a hard surface with a steam road roller. This floor is 610 ft. long and averages 90 ft. wide. No provision was made for expansion and contraction of the steel girders and beams, and there is no evidence of movement in either the steel or the asphalt floor. The expansion of each piece of steel is apparently taken up in the riveted joints. This floor has an inclination of 1 ft. between the end of the train shed and the rear of the head house.

Midway Fence.

The midway fence is a wooden, round-picket fence, 9 ft. high. The posts are fastened at top and bottom. The gates are in two parts, rolling back on the fence, making openings 12 ft. in width, opposite each passenger platform. These passageways are very liberal for crowds, and the fence, taken altogether, finished in natural wood, makes a very neat appearance.

Train Indicators.

Upon this fence, against posts set for the purpose, opposite the ends of each of the twenty-eight tracks, are the train indicators, 6 ft. wide and 9 ft. high; all but two carrying four columns of station names, amounting to about seventy-four names for each indicator. They also give the track number, name of the road, and the leaving time, and are worked by a sort of interlocking machine. Each train is represented by a card, perforated for all stations except those at which the train stops. This card is inserted in the machine and raised with a lever, moving the rods (except at the perforations), which turns the slats on the indicator. These indicators were made by the Wheeler and Wilson Manufacturing Company, Bridgeport, Conn., and are the largest ever made for the purpose.

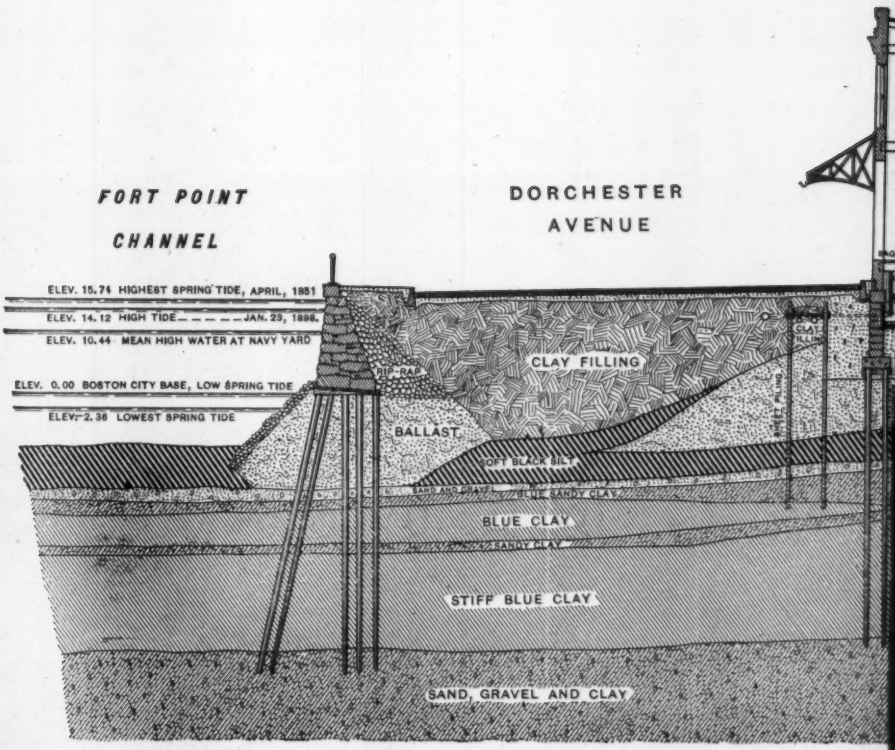
Train-Shed Steel.

The train shed proper is a rectangular building 602 ft. long and 570 ft. wide, divided into three spans, 169 ft. 9 ins., 228 ft. 6 ins. and 169 ft. 9 ins., respectively. The trusses are cantilevers, and this form reduces the connecting trusses in the middle span to 145 ft. $6\frac{5}{16}$ ins. The irregular-shaped areas between the train shed and head house (irregular on account of the lay of the streets), both on the end and sides, are covered by connecting roofs, that at the end being called the midway roof.

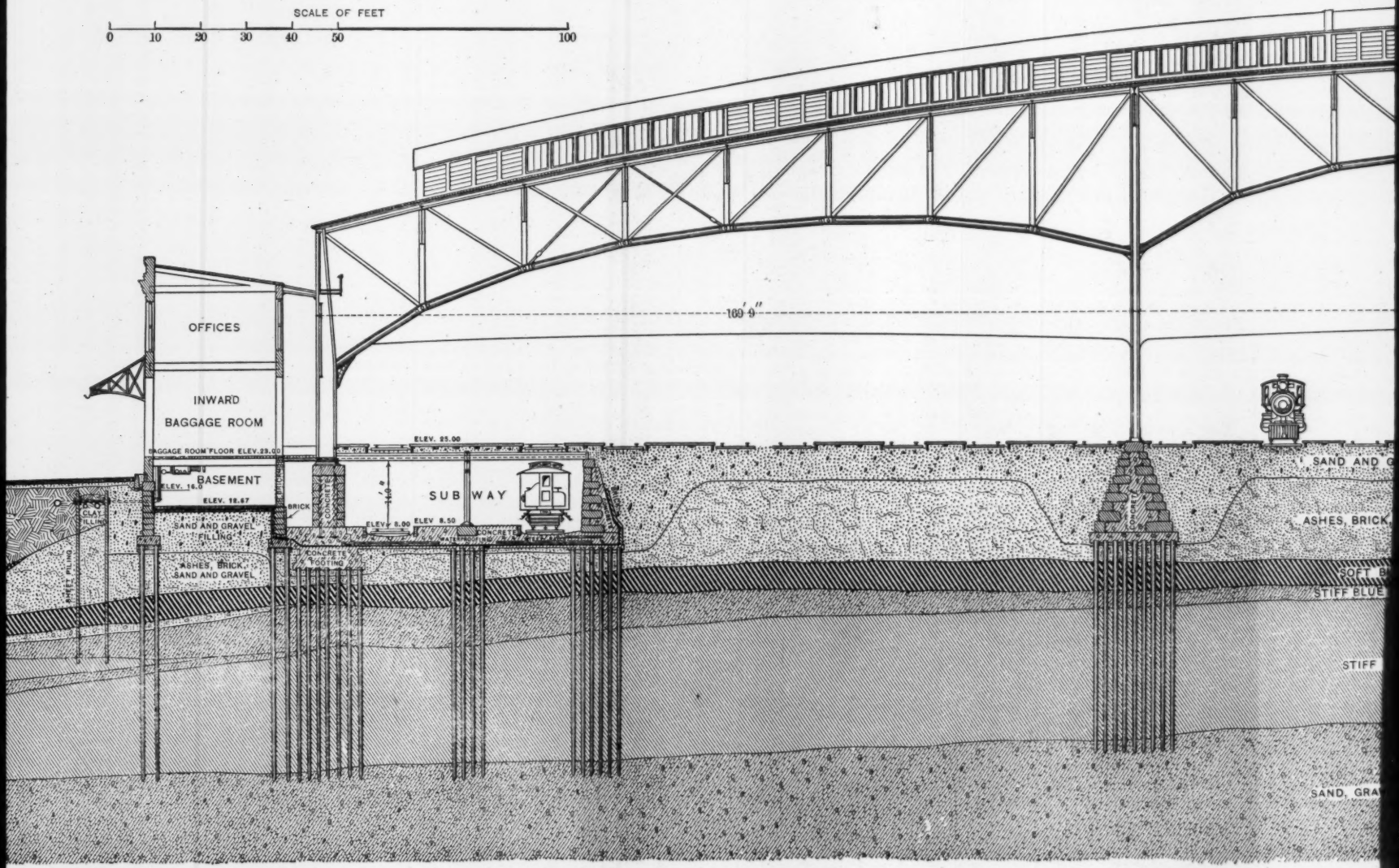
The midway roof and the other connecting roofs are not as high as the eaves of the train shed proper, and, therefore, there is a row of high windows on the sides of the train shed for the entire length. Both gable ends of the train shed are filled with windows. There is a central longitudinal main monitor 60 ft. wide, in the sides of which are both windows and louver ventilators, and on top of which there

TRANSVE

0 1


FORT POINT
CHANNEL
DORCHESTER
AVENUE
ELEV. 15.74 HIGHEST SPRING TIDE, APRIL, 1851
ELEV. 14.12 HIGH TIDE - - - - - JAN. 23, 1898.
ELEV. 10.44 MEAN HIGH WATER AT NAVY YARD
ELEV. 0.00 BOSTON CITY BASE, LOW SPRING TIDE
ELEV. -2.38 LOWEST SPRING TIDE
RIP-RAP
CLAY FILLING
BALLAST
SOFT BLACK MUD
SAND AND GRAVEL
BLUE SANDY CLAY
BLUE CLAY
SANDY CLAY
STIFF BLUE CLAY
SAND, GRAVEL AND CLAY

TRANSVERSE SECTION THROUGH STATION.



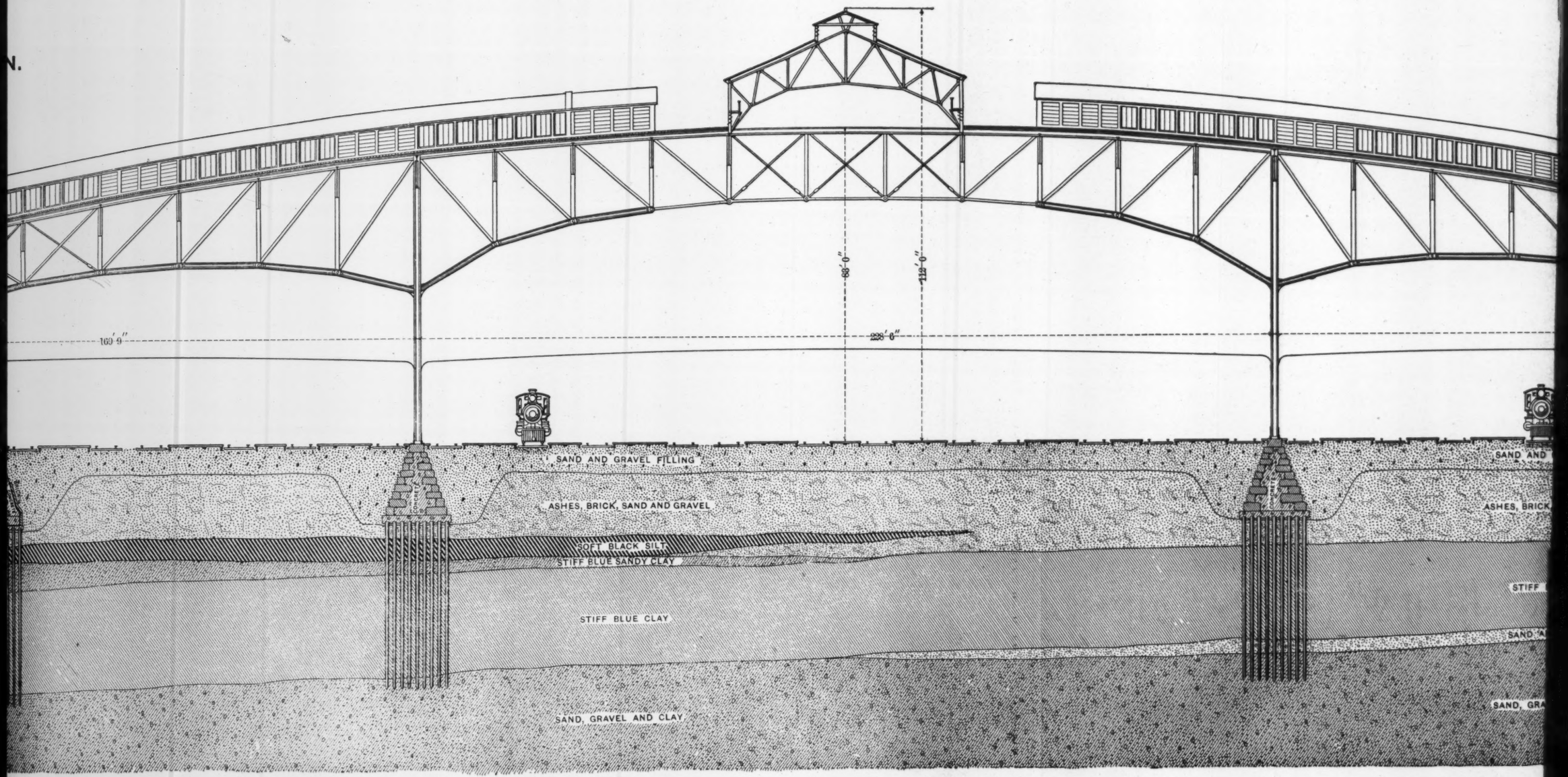
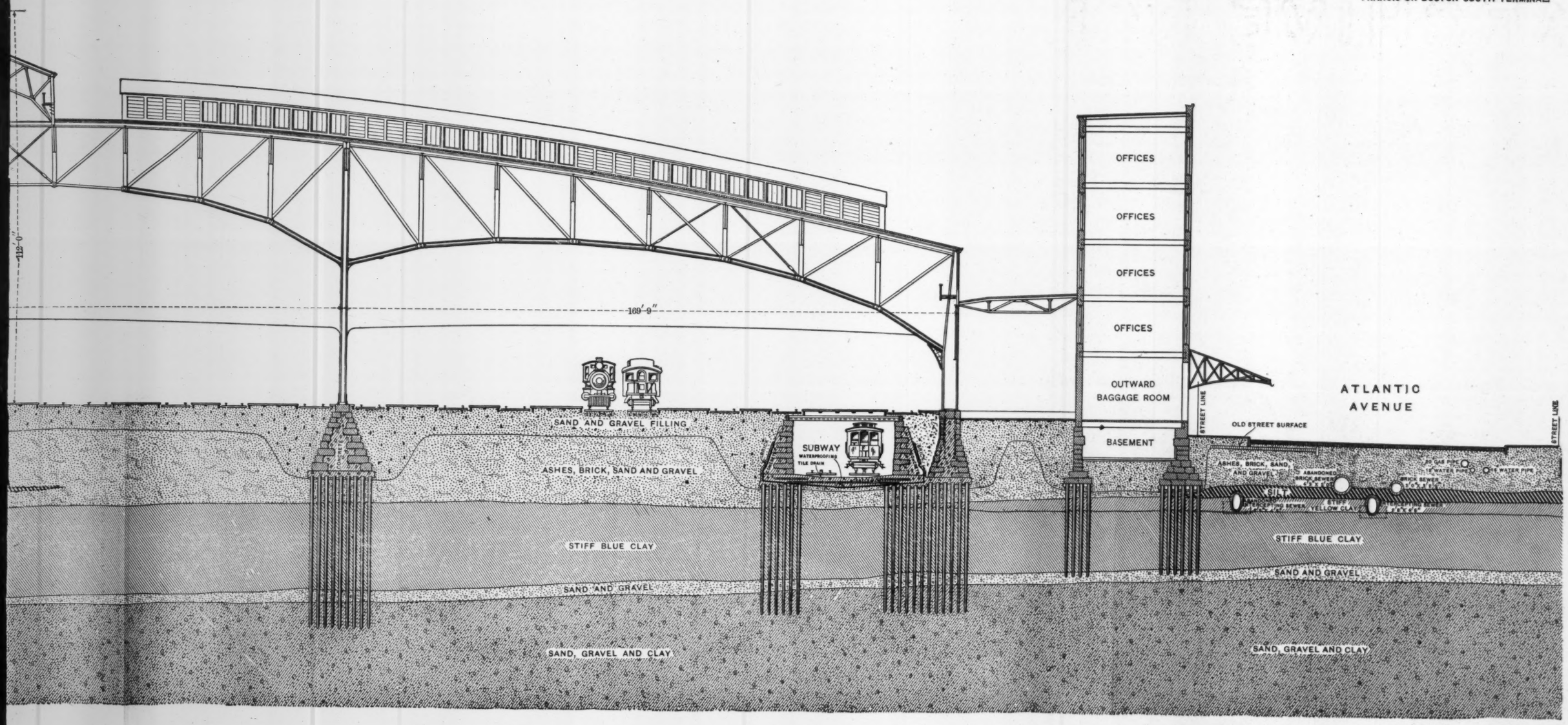

88'-0"
115'-0"
169'-9"
228'-0"
SAND AND GRAVEL FILLING
ASHES, BRICK, SAND AND GRAVEL
SOFT BLACK SILT
STIFF BLUE SANDY CLAY
STIFF BLUE CLAY
SAND, GRAVEL AND CLAY
CONCRETE
CONCRETE

PLATE XIV.
TRANS. AM. SOC. CIV. ENGRS.
VOL. XLIII, No. 870.
FRANCIS ON BOSTON SOUTH TERMINAL.



is a louvered ventilating monitor 16 ft. wide. Over each intermediate truss, on each slope of the main roof, are cross-monitors about 14 ft. wide, reaching from near the eaves to the main monitor. These cross-monitors have windows and also louvered ventilators in their sides, alternating in the various panels. There are no windows or skylights on the slope of the roofs or monitors, all glass being set on the vertical.

The columns and trusses are placed 60 ft. apart, center to center, longitudinally of the shed. The side columns are in box form, and are anchored to the masonry to resist wind pressure. The intermediate columns are not thus anchored, except at the end panels, but carry the major part of the weight of the roof.

The cantilever type for intermediate trusses permitted the making of an arch form to the various spans without unduly increasing the weight of the trusses. This arched form could not have been obtained without the cantilever principle, except at considerable expense for extra weight of metal. All the train-shed trusses are pin-connected.

At each panel point in the trusses is inserted a vertical web member, against which are fastened the purlins, so that all the purlins, which are riveted triangular trusses about 7 ft. 9 ins. deep and 20 ft. apart, stand in a vertical position. Upon these purlins are small I-beam jack-rafters, 7½ ft. apart.

Beginning at each end of the shed, the trusses and columns are braced longitudinally in pairs, until the center is reached, where three trusses and columns are braced together. At the end pairs, the longitudinal bracing is brought to the ground. The end trusses are not cantilevers. The longitudinal expansion and contraction is taken care of between the braced pairs of trusses. The transverse expansion and contraction is taken care of in the intermediate trusses, at the end of one of the cantilevers, on a hinged post, acting as a vertical link; and at the end trusses by supporting the foot of the inclined end post by eye-bar hangers inside of one of the intermediate end posts, the movement in the roof covering taking place at the top of the inclined end post, which is on a line with the ends of the cantilevers. The roof covering is made tight at the expansion joints by simply lapping the higher portion of the roof covering over the lower portion. On the sides and top of the cross-monitors, and at the ends of the shed, a similar lap in the covering is made. At the lower chords of the end

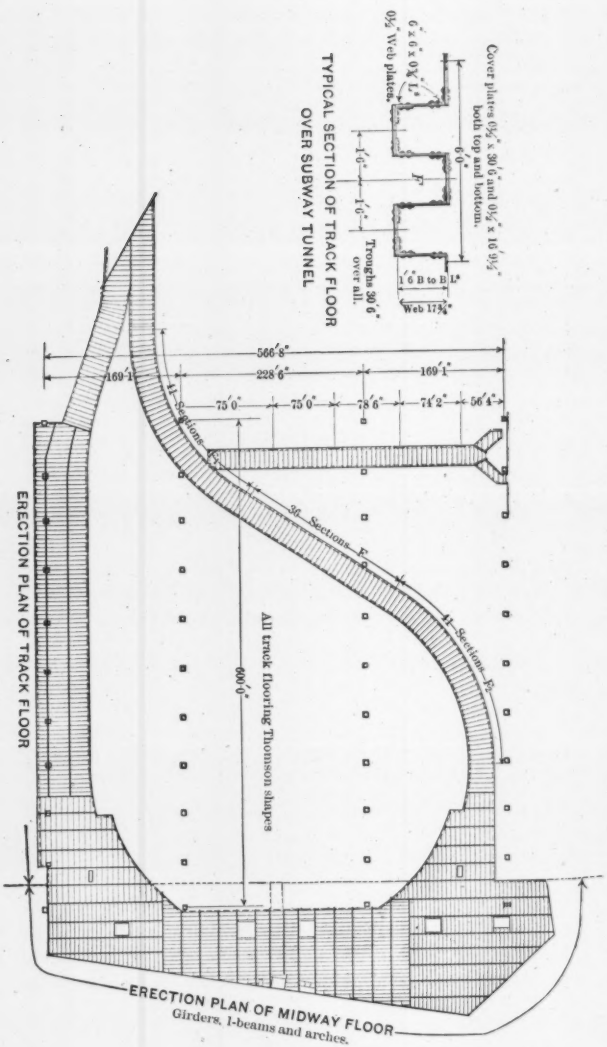
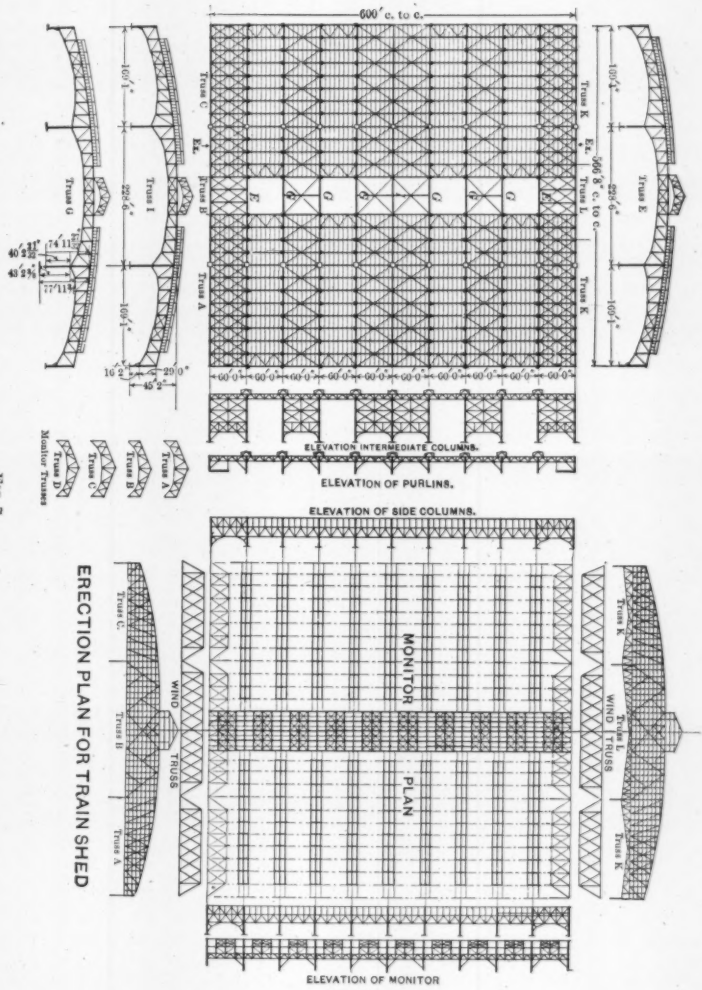

TYPICAL SECTION OF TRACK FLOOR
OVER SUBWAY TUNNEL
Cover plates 0½" x 30'6" and 0½" x 16'9½" both top and bottom.
0½" Web plates.
6" x 6" x 0¾" L's
6'0"
1'6"
1'6"
F
Troughs 30'6" over all.
16 B to B L's
Web 17¾"
566'8"
228'6"
169'1"
169'1"
75'0"
75'0"
78'6"
74'2"
56'4"
41 Sections
36 Sections F
41 Sections F
All track flooring Thomson shapes
600'0"
ERECTION PLAN OF TRACK FLOOR
ERECTION PLAN OF MIDWAY FLOOR
Girders, I-beams and arches.

FIG. 6.



ERECTION PLAN FOR TRAIN SHED

FIG. 7

trusses there are large horizontal wind trusses with latticed members. The gable ends are also braced by and from the end purlins.

The midway roof is anchored at the head-house end of each truss, and is supported at the train-shed end by eye-bar hangers, indirectly from the lower chord. As these midway trusses are square to the head house and spaced at irregular intervals to suit the pilasters of the head house, they do not come at the panel points of the end train-shed truss, and a stiff horizontal member is placed along under the lower chord of the end truss and fastened to it at the panel points, to give support to the midway truss-hangers at any point.

The conditions which governed the architecture of the midway roof were as follows:

A flat-iron shaped area, with a maximum and minimum width of 130 and 50 ft., respectively; a demand that it should not be high enough to shut out light from the end of the train shed or from the offices in the head house; so arranged that light and ventilation should be had for the main waiting-room; and without any post supports in the midway. It appeared to be

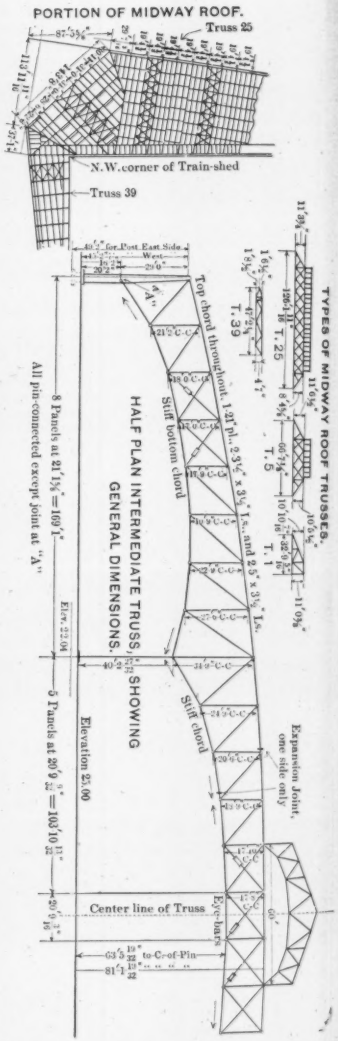
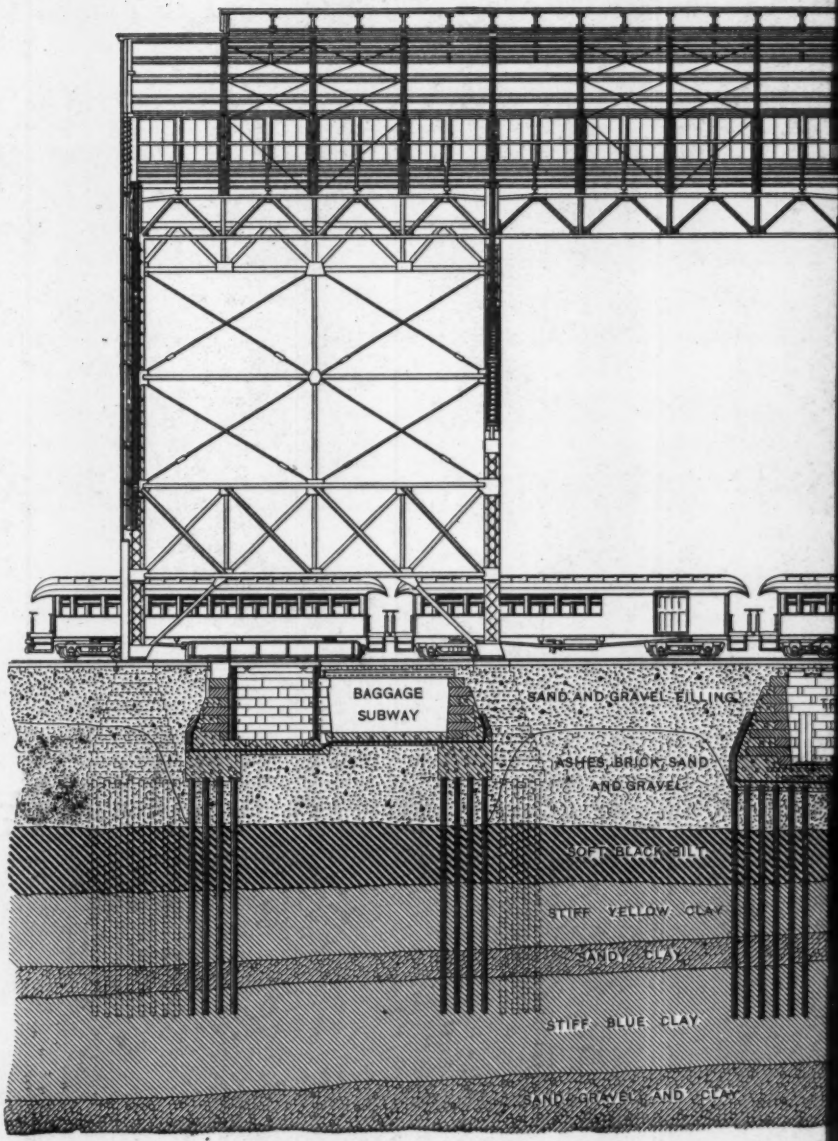
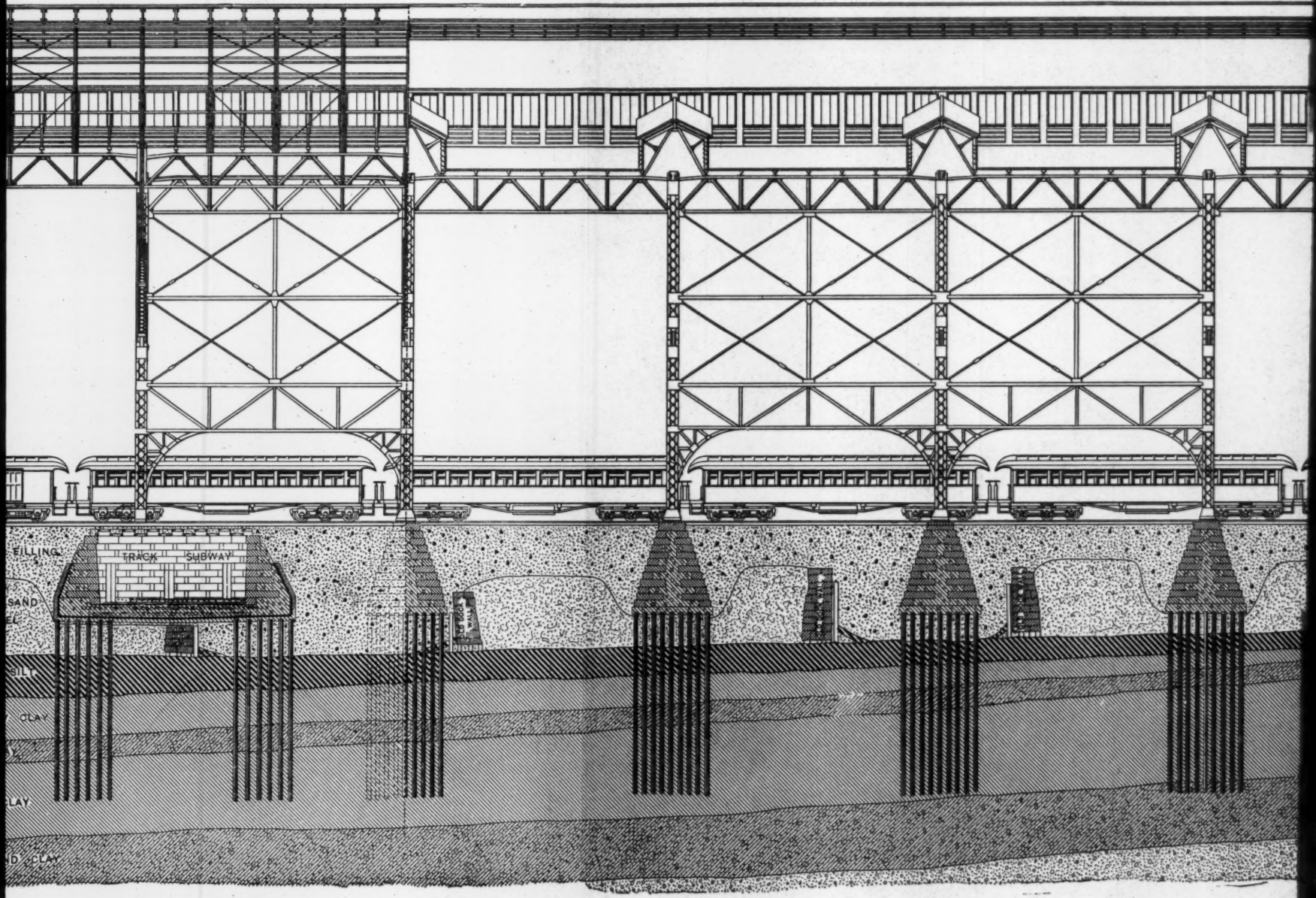


FIG. 8.

SECTION ON CENTER LI



TER LINE. SECTION ON INTERMEDIATE COLUMNS



CTION ON INTERMEDIATE COLUMNS

SECTION ON CENTER LINE

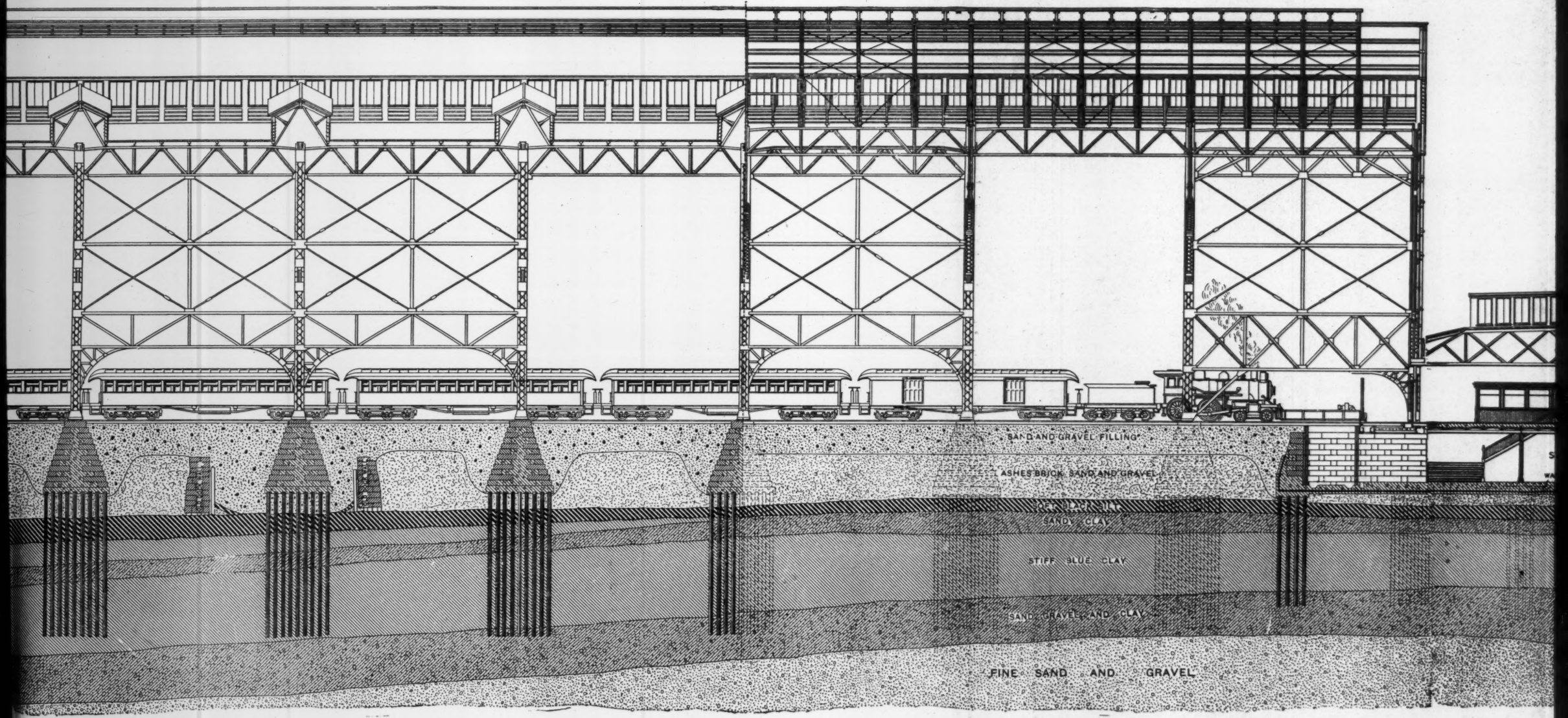
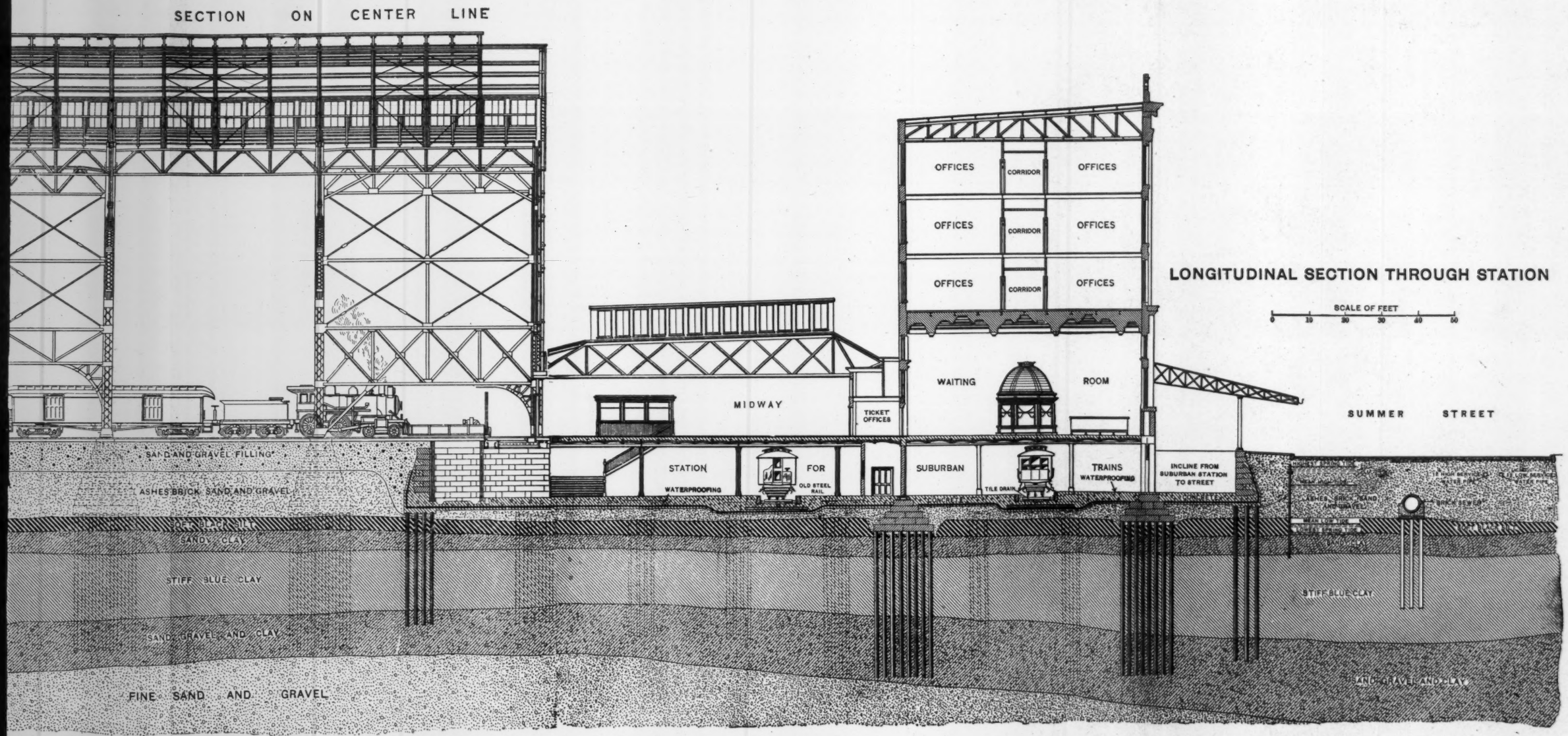


PLATE XV.
TRANS. AM. SOC. CIV. ENGRS.
VOL. XLIII, No. 870.
FRANCIS ON BOSTON SOUTH TERMINAL.



necessary, therefore, to adopt a depth of truss which would be suitable for a 50-ft. span, or a 130-ft. span. This resulted in a riveted-lattice truss, with the bottom chord as high as possible, governed by the windows of the waiting-room, and in order to raise the apparent head-room as much as possible, high monitors were placed above the top chords, in the sides of which were placed the windows, which light and ventilate the midway thoroughly. The low appearance of the midway roof has been the subject of some criticism, but in order to meet the practical demands of the surrounding parts of the station, no great variation could be made, other than perhaps a less number of trusses, more purlins, and a more ornamental treatment.

All the connecting roof trusses are of the riveted-lattice type.

The specifications for the steel in these roofs and the flooring was, in brief, as follows :

Open-hearth steel, to contain not more than 0.06% in basic, or 0.08% in acid, of phosphorus, nor 0.05% of sulphur. Ultimate tensile strength to range from 52 000 to 60 000 lbs. Elastic limit not less than 50% of ultimate strength. Minimum elongation of 26% in 8 ins., and a minimum reduction of area of 50 per cent. Rivet steel to have an ultimate tensile strength to range from 50 000 to 56 000 lbs., with minimum elongation of 30% in 8 ins., and minimum reduction in area of 60 per cent. For bending tests, the usual requirements for bridge specifications were insisted upon. All the steel was inspected at the rolling mill, and found to be of good quality, calling for very little rejection. The work was oiled and painted with one coat of red lead and oil at the shop, and has had another coat of red lead and oil since erection.

The thankless and hopeless task of selecting a final color, and a paint which will preserve the metal intact and at the same time be beyond all criticism, has not yet been undertaken.

The specifications for loading on the roof and midway floor and tracks were as follows :

First.—Roofs.

Vertical; the dead weight, plus 25 lbs. per square foot of roof surface.

Horizontal; a wind force of 30 lbs. per square foot of vertical surface.

Second.—Midway Floor.

The dead weight, plus 150 lbs. per square foot.

Third.—Tracks.

Dead weight, plus a rolling load for a single-axle load of 50 000 lbs., or axle loads as follows for weights and spacing:

14 000 lbs., 40 000 lbs., 40 000 lbs., 40 000 lbs., 40 000 lbs., 22 000 lbs., 22 000 lbs., 22 000 lbs., 22 000 lbs.; 7′ 0″, 5′ 6″, 4′ 6″, 5′ 6″, 12′ 0″, 5′ 0″, 5′ 6″, 5′ 0″.

Allowed unit strains were, in brief, as follows:

First.—Roofs.

Tension, eye-bars, 16 000 lbs. per square inch.

Tension, built sections and counters, 15 000 lbs. per square inch.

Tension, laterals, 18 000 lbs. per square inch.

Compression, in pounds per square inch:

$$\text{Fixed ends, } \frac{12\,000}{1 + \frac{L^2}{36\,000\,r^2}}$$

$$\text{Hinged ends, } \frac{12\,000}{1 + \frac{L^2}{18\,000\,r^2}}$$

Combined fiber strains from bending and direct strain, and combined direct strain from vertical and horizontal bending.

Tension, 18 000 lbs. per square inch.

Compression, in pounds per square inch:

$$\text{Fixed ends, } \frac{14\,000}{1 + \frac{L^2}{36\,000\,r^2}}$$

$$\text{Hinged ends, } \frac{14\,000}{1 + \frac{L^2}{18\,000\,r^2}}$$

Second.—Midway Floor.

Tension, 15 000 lbs. per square inch.

Compression in columns, in pounds per square inch:

$$\frac{12\,000}{1 + \frac{L^2}{36\,000\,r^2}}$$

Compression flanges, same as tension.

Shear in webs, 9 000 lbs. per square inch, or

$$\frac{12\,000}{1 + \frac{d^2}{3\,000\,t^2}}$$

FIG. 1.—FOUNDATIONS AND WATERPROOFING NEAR MAIN ENTRANCE.

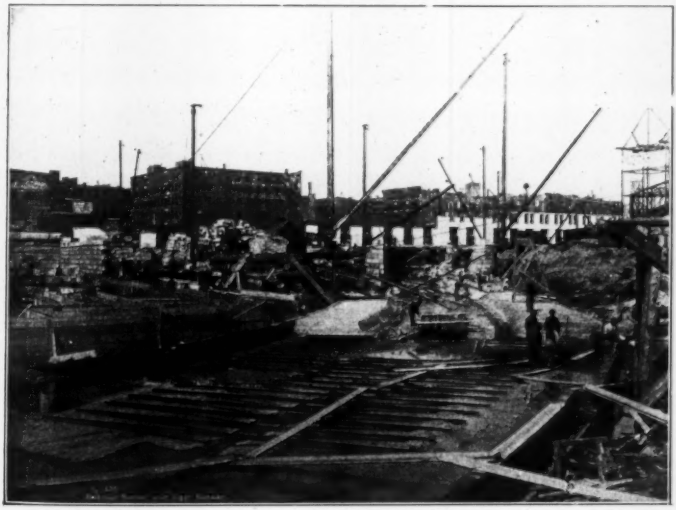


FIG. 2.—BAGGAGE SUBWAY AND WEST SUBWAY.

Third.—Tracks.

Tension, 9 000 lbs. per square inch.

Compression in columns, in pounds per square inch:

$$\frac{8\,000}{1 + \frac{L^2}{36\,000\,r^2}}$$

Compression flanges, same area as tension.

Shear in webs, 7 500 lbs. per square inch, or

$$\frac{10\,000}{1 + \frac{d^2}{3\,000\,t^2}}$$

Bending strains in pins, 20 000 lbs. per square inch.

Bearing on pins, 18 000 lbs. per square inch.

Bearing on rivets. Enclosed bearings, 18 000 lbs. per square inch in the roof and midway floor; 15 000 lbs. per square inch under tracks. Unenclosed bearings, 15 000 lbs. per square inch in roof and midway floor; 12 000 lbs. per square inch under tracks. On field rivets, the above strains were decreased 25 per cent.

The workmanship was required to be similar to that called for in the usual bridge specification.

The work was accurately punched and not reamed.

Copies of all shipping invoices were forwarded to the engineer of the Terminal Company, and the actual scale weights at the shop were checked by calculated weights by the Terminal's Company's engineer. On the total of about 20 000 000 lbs. of steel, these weights agreed within about $\frac{1}{4}$ of 1 per cent. Payments were made on the scale weights, as made at the shop. Monthly payments were made on the material received and erected, and on material delivered on cars in the storage yard at Boston, when consigned to the Terminal Company and not to the contractor.

Train-Shed Covering.

The main roof and side covering is a 2-in. tongued and grooved hard pine sheathing, in 8-in. widths, nailed with iron wire-nails to strips bolted to the jack-rafters. This sheathing, upon the main sweep of the roof and on the tops of the monitors of the midway roof, is covered with "Warren's" anchor brand composition roofing, made up as follows:

Two thicknesses of double roofing felt, manila side down; two thicknesses of natural asphalt roofing felt, each sheet mopped under its full width and upon the top surface with natural asphalt roofing cement, upon which is laid a sufficient body of well-screened dry white Long Island beach gravel.

The tops of all monitors on the train-shed roof and the main portion of the midway and connecting roofs, as well as all side and end sheathing, are covered with 16-oz. copper, as are also all of the exterior main parts of the window-frames in the ends and sides of the train shed and the sides of the monitors. All flashing, cornices, gutters and down-spout connections are of copper.

The train shed and midway are lighted through asbestos-covered wire-glass, bedded in putty, fastened in with wooden strips in wooden sash, fastened with brass screws to wooden frames bolted to the train-shed steel. All the glass is set vertically, and permanent foot-walks have been provided, so that it will be possible to reach each pane of glass in the shed at any time with brushes and water to keep it reasonably clean. The glass is ⅛-in. thick.

It is not intended that it shall be necessary to shovel any snow from the main roof. All the down-spouts, which are 8 ins. in diameter and generally 60 ft. apart, are covered with jackets, and steam pipes have been run between the spouts and the jackets to keep them from freezing in the coldest weather.

The ventilators are made of timber, are of ordinary louver pattern, and are distributed quite uniformly throughout the monitors. Ventilation over the midway is obtained by opening some of the windows in the monitors. All the woodwork is covered with three coats of white lead and oil paint, the finishing color being on the outside a dark green or copper color, and on the inside a medium drab.

It is believed that this is the first instance where foot-walks have been provided on such an extensive scale, practically to every window, in any train shed, and where the lighting is done exclusively through vertical windows. The result has been very satisfactory.

Head House.

The main entrance to the station is at the intersection of Federal Street, Summer Street and Atlantic Avenue, and it is here that the main architectural features of the station are found.

FIG. 1.—NEW CITY SEWER, STRAPPED PORTION.

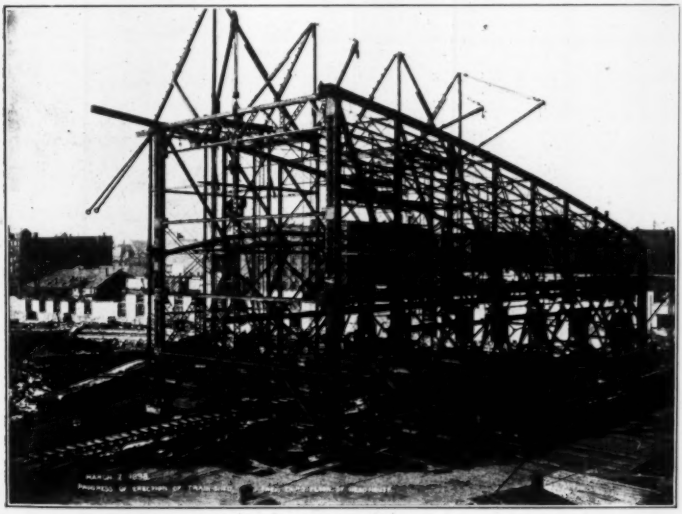


FIG. 2.—PROGRESS OF ERECTION OF TRAIN SHED.

The building extends from this entrance, 792 ft. along Atlantic Avenue (formerly Cove Street), and along Summer Street 672 ft.; then turning the corner of Dorchester Avenue, it extends 725 ft. further, making the total street frontage of the head house 2 189 ft. Two stories, for this entire length, are given up to station purposes, and the three upper floors of the five-story building are used for office purposes by the operating railroad companies.

The five-story building, or main office building (in the middle of which is the main entrance), is 875 ft. long, of which 228 ft., or the portion at the main entrance, is curved.

Of the curved portion, two stories form a massive base, in which are the three large arches forming the entrance. The upper three stories are treated as a colonnade. There are sixteen of these columns, 4½ ft. in diameter, and 42 ft. high. These columns support an entablature and parapet, with a projecting pediment over the center. Above this pediment is a clock, with a dial 12 ft. in diameter, in an elaborate granite setting. Over the clock is a large granite eagle, with wings partly spread, stooping as if just ready for flight. This eagle is about 8 ft. high, and the same breadth over the wings.

In front of the building, opposite the center of the main entrance, there has been erected an ornamented polished granite column, upon a heavy polished granite base, to carry five large electric lights. This column is about 40 ft. high.

The curved portion of the building is of cut granite, and nearly all the remaining front of quarry-faced granite, laid in courses. Upon the front of each wing of the five-story building there are large panels of buff brick, which relieve the severe appearance of the granite.

There is a secondary entrance to the station from Atlantic Avenue, also an exit from the subway. The remainder of the front on this avenue is devoted to the outward baggage-room, the doorways being protected by an iron and glass awning, extending out sufficiently to cover all baggage in transit from the wagons to the building.

On the Summer Street side there is a series of large arched windows to give light to the waiting room; beyond these is the main exit, a wide passageway leading directly from the midway to the street and passing over the subway inclines by a bridge. These inclined subway exits are below the ground-floor level, and lead up to the street from the subway platforms, avoiding the use of steps.

The carriage concourse is at the corner of Dorchester Avenue and Summer Street. Opposite the train shed, on the Dorchester Avenue side, is the inward baggage room, about 550 ft. long. Here, again, the doorways and teams are protected by an awning. The awning along Summer Street is about 40 ft. wide, and protects the subway exits as well as the sidewalk.

The sidewalks all around the station are lighted by the Terminal Company from their own electric plant.

The main entrance is a thoroughfare 90 ft. wide, lined on each side with polished Stony Creek granite. Within the entrance are four large polished granite columns, about 3½ ft. in diameter, which support the offices above. In the polished side walls are cut the date of the erection, the names of the constituent railroad companies, and the trustees of the terminal company, also the names of the mayor, engineers, architects and builder. The ceiling is of white enameled brick, and the iron beams are enclosed in white marble.

Opening from the midway, on the right, are the parcel room, entrance to elevator and stairway hall leading to the offices, and the outward baggage room. On the left are toilet rooms, telegraph, telephone, ticket offices and information bureau, separated by openings to the waiting room. The ticket office has eleven windows opening on the midway and sixteen on the waiting-room side.

The waiting room is 225 ft. long, 65 ft. wide, and 28½ ft. high, and out of the line of traffic. The floor is marble mosaic, laid with a large and handsome pattern. The sides have a dado of enameled brick, set on a polished granite base, and above this the walls are of plaster—tinted. There are three polished granite doorways, and two Verd-antique drinking fountains. The room is lighted during the day time from windows on Summer Street and also from windows above the midway roof (which was kept low partially for this purpose), and by night from 1 200 incandescent lamps distributed along the side walls and in the deep modeled stucco coffered ceiling, giving a beautiful, unobtrusive and well-diffused light. Along one side are arranged great oak settees, placed to form alcoves. On the middle axis of the room are two large ornamental kiosks for the sale of confectionery and flowers.

The women's waiting room is entered from one corner of the main waiting room. This room is 34 by 44 ft., and is furnished with rock-

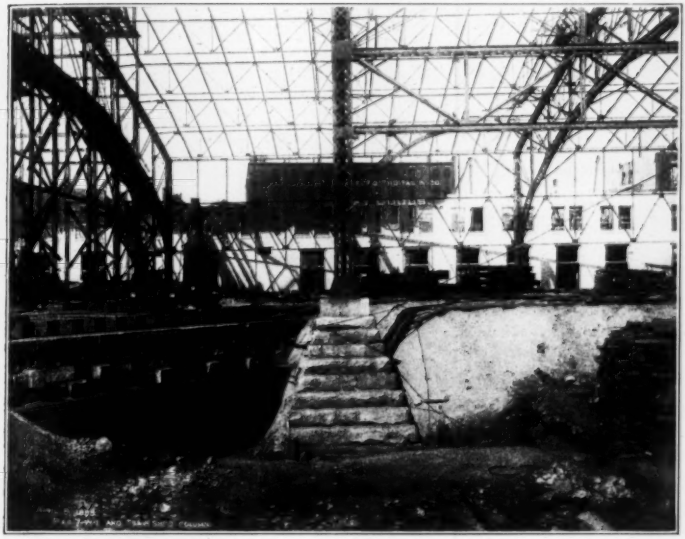


FIG. 1.—PIER AND TRAIN-SHED COLUMN.

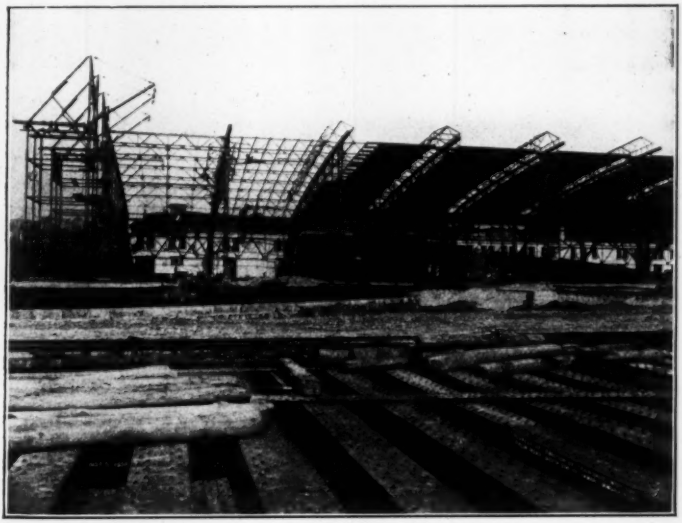


FIG. 2.—PROGRESS OF ERECTION OF TRAIN SHED.

ing chairs, lounges, tables, cribs and cradles. Adjoining are ample free and pay lavatories.

At the easterly end of the waiting room is the main exit and stairway to the subway, also an elevator to the offices and subway. Beyond the exit is the lunch room, 67 by 73 ft., with marble mosaic floors and side walls similar to the waiting room. Here are about 200 stools at the lunch counters, which latter are made with Tennessee marble face and mahogany tops. Next comes the serving room, also the elevator and stairway to three large dining rooms on the second floor. The kitchen and other serving rooms are also on the second floor. Near this last elevator, on a mezzanine floor, is an emergency room, with proper instruments and attendants for giving first aid to the sick or injured. Following along the midway are the station master's office, barber shop, shoe-polishing room, public lavatory, smoking room and carriage and transfer office, and at the extreme end of the midway is a passage to the inward baggage room.

In the midway are arranged five large booths, for the sale of newspapers, fruit, tobacco, drinkables, and for the rendezvous of the baggage porters.

Below the main floor are rooms for baggage storage, emigrants and restaurant purposes.

The second floor is occupied by the administration offices of the Terminal Company, and the trainmen.

The third floor is occupied by the main offices of The Boston and Albany Railroad Company, and the fourth and fifth floors by the local offices of The New York, New Haven and Hartford Railroad Company.

There are nearly 100 clocks throughout the buildings, all self-winding, and governed by electric connections from the master clock in the station master's office. There is also installed a system of watchman's magneto-alarm clocks, a magneto-box being placed near each treasurer's vault, as well as at other suitable points.

All the departments are supplied with ample fire-proof vault accommodation, the largest being 44 x 24 ft., and used to hold engineers' plans. Fire-alarm boxes and push-buttons are arranged at suitable points and connected with the city's fire-alarm system, and movable hose reels are also installed at various places about the premises.

Norcross Brothers, of Worcester, Mass. (O. W. Norcross, proprietor), were the contractors for the entire head house, including the foundations, the foundations for the train shed, the train shed and connecting roof coverings; also for the express and power buildings, and nearly all the subway foundations and masonry.

Plumbing.

Throughout the terminal buildings there are about 45 toilet rooms, fitted up with first-class apparatus, generally known under the trade name of "Sanitas" fixtures. All the water-closets in the free public lavatory rooms are automatically flushed by the seat movement. The closets in the pay and office lavatories have a chain pull. There are upwards of 200 water-closets. The urinals are automatically flushed at adjustable intervals. Nearly all fixtures are connected with the ventilating fans in the attics.

In all the principal toilet rooms there are porcelain slop sinks, fitted with hot and cold water, as well as an ample supply of wash bowls and mirrors. For the train men there are several shower baths.

Each toilet room has an asphalt floor, and a pail and broom closet.

The restaurant plumbing is equipped with large grease traps, and there is a thoroughly ventilated swill room, where the swill buckets, after being emptied, can be cleansed with hot water upon the floor. This room being below the street sewer level, the wash water is lifted by means of a "Shone pneumatic ejector."

Water Supply.

The water used throughout the terminal is taken from the city water mains. For the fire supply, from the high-service pipes, it enters the building at five different places, and a complete outfit of standpipes and hose is established from basement to attic. For the general supply, the water is taken from the high-service mains for the five-story building and from the low-service mains for the other buildings. A 12-in. main is laid clear across the yard near the end of the train shed, from which a supply is obtained for locomotives, through stand pipes, and through which a circulation is maintained, so that it is practically impossible to deprive the power house of water. The general supply enters the premises at fifteen different

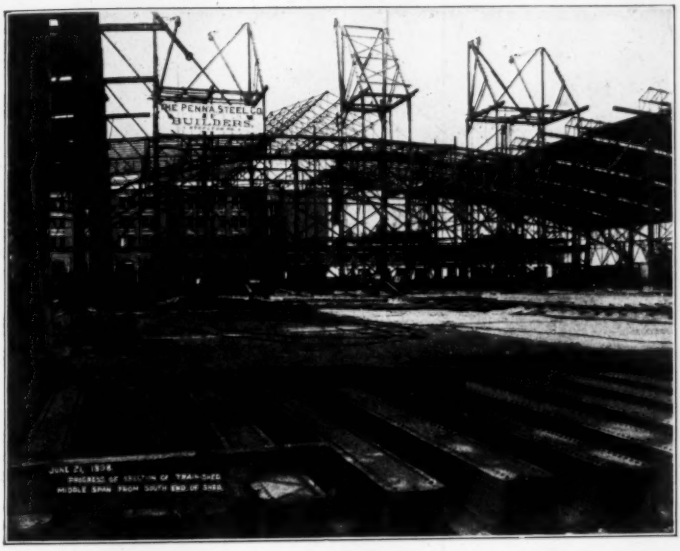


FIG. 1.—PROGRESS OF ERECTION OF TRAIN SHED.

FIG. 2.—VIEW THROUGH MAIN MONITOR OF TRAIN SHED.

places, and at each point the consumption is determined weekly by meters set for the purpose, and any excessive use quickly located and corrected. There are also several other meters to measure water supplied to tenants in the express buildings and restaurant.

Present indications show a probable annual expense for water in the neighborhood of $9 000. No actual trials were made to obtain water from driven or artesian wells, but, judging from the experience of others not far away, it was not expected that it could be thus obtained without being slightly brackish.

Interlocking.

The system of interlocking adopted at the South Terminal Station is what is known as the Westinghouse electro-pneumatic.

Previous to the selection of this system, plans were drawn to show the adaptability of both the mechanical and the electro-pneumatic systems. Owing to the demand at this station for a large and elaborate track layout, to serve the 737 trains (total of inward and outward) which would use it immediately upon its completion, and the fact that this number of trains would create many more times the number of train movements than 737, it required a larger interlocking plant than had been heretofore operated anywhere from one tower. When the plans were drawn for a mechanical plant, it was found that a very large building was required, and that a very large area of valuable land would be taken for the lead-out piping, the width required, near the tower, in each direction, being about 45 ft. No suitable way could be found to place this piping in a vertical position. It was also plain that a very much larger number of men would be required to operate the mechanical plant than the electro-pneumatic, and that it would be difficult to make such rigid foundations for the pipe lines on the new fill over the old docks as they require, without waiting a great many months for the banks to settle. It also appeared that, owing to the very magnitude of the plant, which called for unusual treatment, it would cost as much, or perhaps more, to establish the mechanical as the pneumatic. So that even with the remote possibility of electric currents, required in the pneumatic plant, becoming troublesome when electric motive power should be adopted, there seemed to be no other reasonable alternative than to select the electro-pneumatic system.

In the tracks embraced in the control of No. 1 tower are switch and frog points equivalent to 238 ordinary switches. Eleven trains may move to or from the train shed at one time, to control which there are 148 semaphore signals.

In giving a description of the interlocking machine and the electro-pneumatic valves, it will be necessary to quote partially from descriptions by those who are more thoroughly posted in all the details than the writer. Each pair of switch points and each semaphore are attached to the piston of a pneumatic cylinder, which is secured to the switch or signal support. The admission and discharge of air to or from the cylinders is controlled by valves, which in their turn are shifted by electro-magnets, governed by the interlocking machine in which are horizontal rotating shafts, moved by small levers through an arc of 60°, with electric contacts thereon, suited to give the required condition to the magnets at the valves, and thereby produce the proper movement of the switch or signal.

The rotating shafts are arranged to move bars above them, and at right angles to them, which interlock with one another by a system of cross locks. These bars extend from end to end of the machine. The ends of these shafts are engaged by the armatures of electro-magnets, which are so governed, by the switches and signals operated, that the levers and the apparatus operated by them must agree in position before a prescribed route may be given.

A working model is attached to the interlocking machine, showing all the switches moved by the interlocking, corresponding to the movements in the yard, and this model gives a correct representation of the track connections.

There are nine steel-truss signal bridges in the yard, and the signals generally are placed upon them. The semaphore posts are hollow iron columns, with the operating connections inside, and are set, as far as possible, over the center of the track to which they apply. The blades and lights adopted are in accordance with the system of the New York, New Haven and Hartford Railroad, which company operates more than two-thirds of the trains entering the station. The blades extend to the right from the pole. Forked blades are used, to some extent, to indicate that a route clear through the system has been arranged, and at the last signal before entering the train shed to indicate whether or not cars are standing upon the

FIG. 1.—SOUTH END OF TRAIN SHED.

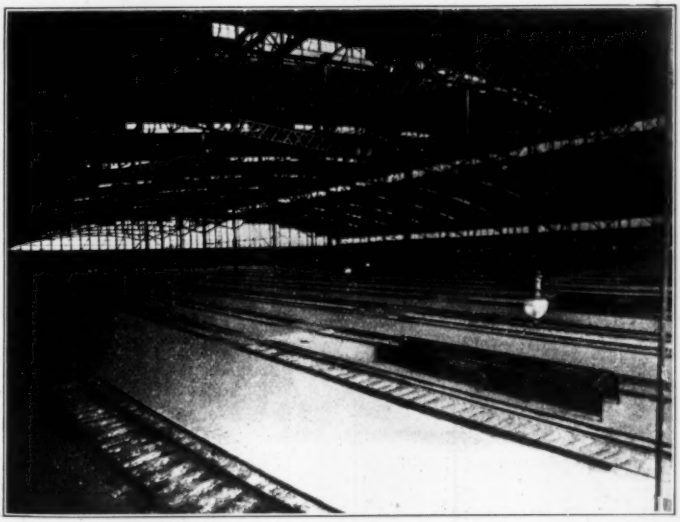


FIG. 2.—INTERIOR OF TRAIN SHED.

track in question inside the shed. Red is used as a stop, green as an all right, and yellow as a caution signal color.

The air cylinders used to move the signal arms are single-acting, the arm being held in a stop position or brought back to a stop position by a counter-weight, and therefore require but one valve.

The air cylinders used to move the switches are double acting, and, therefore, require two pin-valves (one for each end of the main cylinder), controlling auxiliary cylinders, which shift a D-valve, capable of more readily controlling the pressure to and from the switch cylinders. The D-valve cannot act unless a plunger which prevents its movement is first withdrawn. This is effected by a third magnetic valve, and auxiliary cylinder, and the three magnetic valves are controlled by three separate wires extending from them to contacts of the interlocking machine. The D-valve is used on account of the large volume of air required to be admitted in a short space of time.

The first part of the stroke of the main piston moves the detector bar, the middle part moves the switch, and the latter part locks it in position; consequently, if a detector bar cannot rise, owing to being held down by the wheels of a train, the switch cannot be thrown. There are in the power-house two air compressors, which furnish the air power. They are 14 x 18-in. Ingersoll-Sargeant, class "A," compressors. One is held in reserve as a relay to the other. Each compressor discharges the air into receiving tanks, from which it is conducted to a third receiver. The air mains leading from the receiver are duplicated, so that failure in one line of piping will not prevent an immediate operation of the switches and signals through another line. The air passes through cooling devices, consisting of manifold pipes, which precipitate all moisture, and at each switch enters a cast-iron receiver. From this reservoir there is an armored hose connection to the switch valve. This elastic connection prevents injury to the connections, due to vibration and settlement.

There are also provided indicators from the train house to the tower for proper communication, and the towers are equipped with telephone connection to all necessary points.

Tower No. 2 controls switches and signals on the suburban tracks, and Tower No. 3 governs train movements at the yard limits too remote to be controlled from Tower No. 1.

These towers and the other main-yard buildings are of brick, with

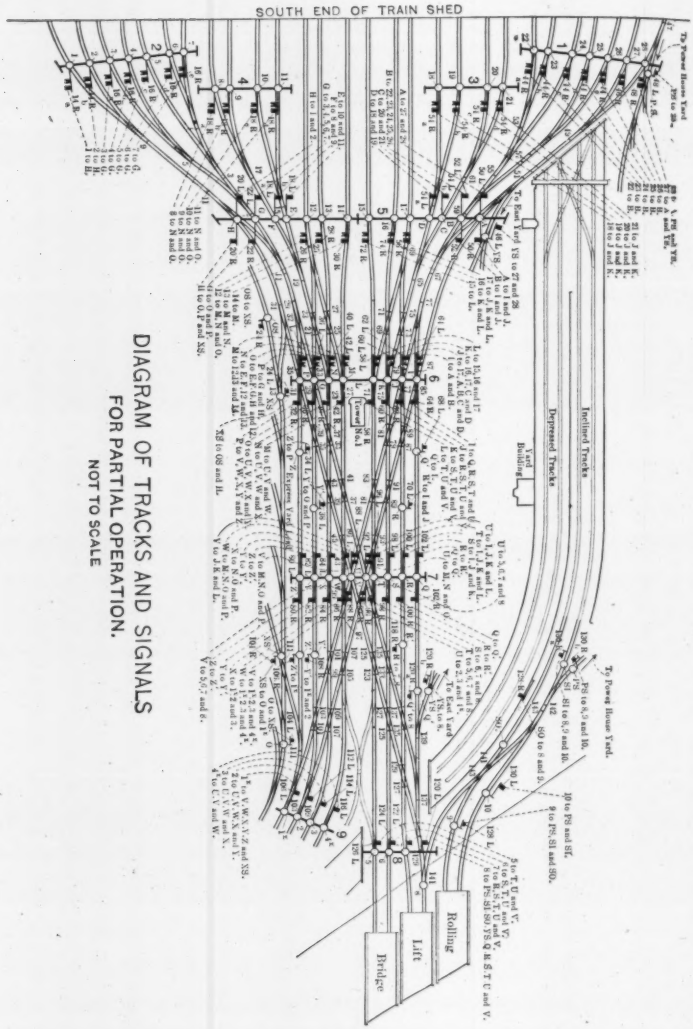


DIAGRAM OF TRACKS AND SIGNALS
FOR PARTIAL OPERATION.
NOT TO SCALE

FIG. 9.

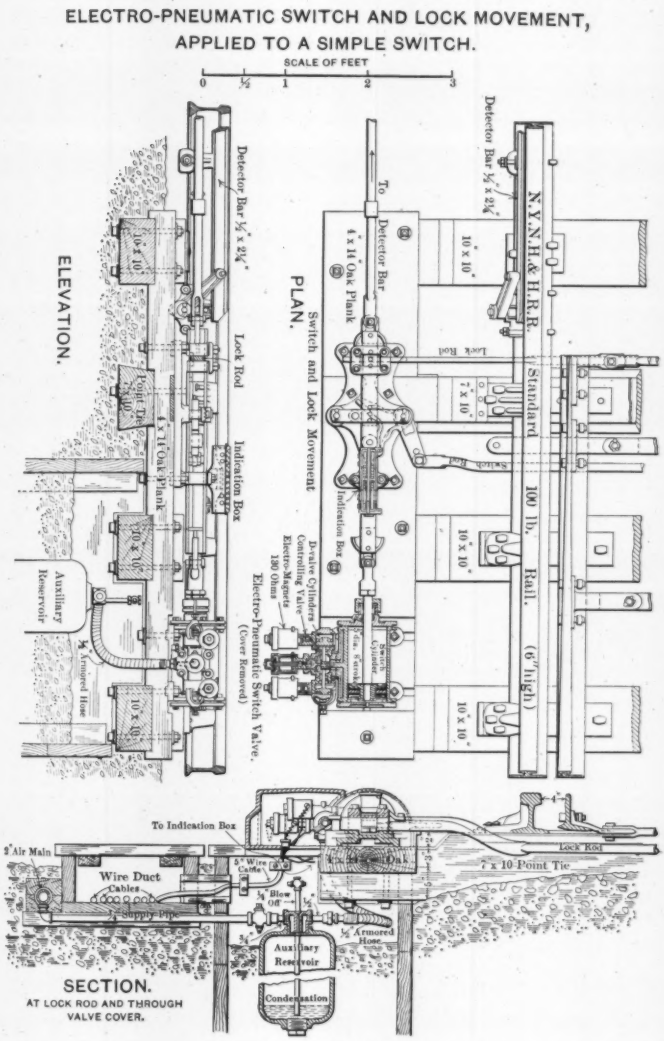

ELECTRO-PNEUMATIC SWITCH AND LOCK MOVEMENT,
APPLIED TO A SIMPLE SWITCH.
SCALE OF FEET
0 ½ 1 2 3
ELEVATION.
PLAN.
SECTION.
AT LOCK ROD AND THROUGH VALVE COVER.
Switch and Lock Movement
Detector Bar ½ x 2¼"
Lock Rod
Indication Box
Point Tie
4 x 14 Oak Plank
Auxiliary Reservoir
Armored Hose
Electro-Pneumatic Switch Valve.
(Cover Removed)
Pneumatic Cylinders
(Controlling Valve
Electro-Magnets
130 Ohms
To Detector Bar
N.Y.N.H.& H.R.R. Standard 100 lb. Rail. (6" high)
10 x 10"
7 x 10"
Switch Rod
Lock Rod
Indication Box
To Indication Box
2" Air Main
Wire Duct
Cables
¾" Supply Pipe
5" Wire Cable
¾" Blow Off
¾" Armored Hose
Auxiliary Reservoir
Condensation
Oak
7 x 10 Point Tie

FIG. 10.

slate roofs, and contain toilet fixtures for the accommodation of the employees.

Signal Lights.

Oil lamps are used for signal lights, excepting on one signal bridge, where experiments are being made with electric lights. These experiments have not progressed far enough to prove conclusively the superiority of electricity, but give evidence of doing so. The trial lights consist of two 8-candle-power lamps in each lantern, so that in case of failure of one, the other may do sufficient duty. They are very clear, and penetrate fog better than the oil lamps.

If the details can be arranged to give satisfaction in all kinds of weather, they will undoubtedly be adopted.

Data are also being collected to determine the comparative cost.

Track Grades.

The tracks in the train shed are level. At the throat of the interlocking they are 2 ft. lower than in the shed, to ease the grade on the curved approach, which is about 40 ft. to the mile (upon 8° curves), and to facilitate reasonable connections with the express-yard tracks, which of necessity lie at a grade 6 ft. lower than the main tracks.

The grade of the tracks on the drawbridge is practically the same as in the train shed.

The steepest grade in the express yard is about 1½ per cent.

The loop tracks are level. The grade upon the inclines leading to the loop tracks does not exceed 2½ per cent.

In the train shed the tracks are level with the platforms; in the mail and express yard the tracks are about 4½ ft. below the level of the platforms, the latter being substantially on a level with the floor of the cars.

Express Buildings, Team and Car Yards.

The express buildings are 50 ft. wide and two stories high. They are owned by the Terminal Company and leased to the express companies. They are similar in style and appearance to the power-plant buildings.

One portion of the express tracks is laid parallel with the building,

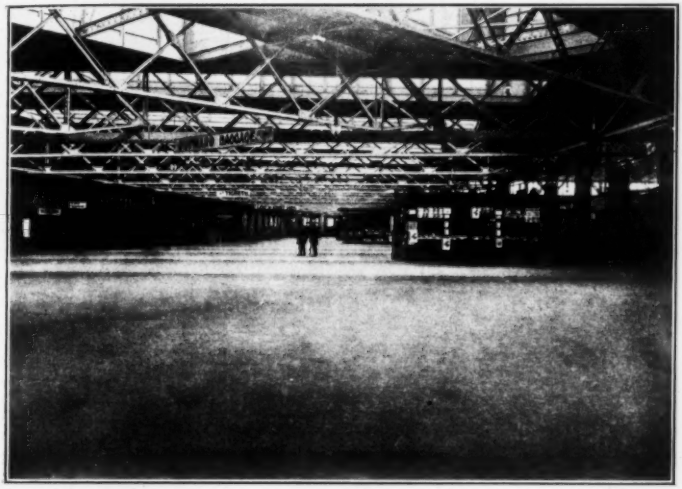


FIG. 1.—THE MIDWAY, LOOKING EAST.

FIG. 2.—SERVICE TRACK CONNECTIONS WITH N. E. R. R.

as requested by the express company occupying that part of the building. The other portions of the tracks are laid as spurs, at an angle to the building, with intervening platforms. This arrangement was necessary to get the room required for cars.

The team side of the building has a covered platform nearly its entire length, and the yard is paved with granite blocks. The second floor is used for offices by the Adams Express Company, and store rooms for the New York, New Haven and Hartford Railroad Company. A portion is unused at present.

Heating and Lighting for Express and Yard Buildings.

The express buildings are heated by hot water, directly through radiators in the various rooms. It is not intended that the freight rooms and store rooms shall be kept much above freezing in the winter, but the offices are thoroughly warmed.

The piping system is entirely separate from the head-house system after leaving the power house, and was installed under a separate contract. The fire-risers through the freight rooms, in this building, as well as those through the baggage rooms in the head house, are boxed and protected from the frost. The platforms and freight rooms are lighted by arc lights, and the office rooms by incandescent lights. Two of the five electric truck lifts go to the store rooms in the second story, and these two are arranged with safety clutching devices.

The yard buildings are heated by steam and lighted by electricity.

Power Plant Buildings and Pipe Subways.

The power-plant buildings are substantial hard-burned brick buildings, with granite trimmings and flat gravel roofs upon steel trusses. They are 40 ft. wide and two stories high, with an aggregate length of 580 ft.

The pipe subways leading from the power plant to the head house and express buildings are 6 ft. wide and 8 ft. high, and are built entirely of Portland-cement concrete, as shown in Fig. 4. They carry the pipe lines on one side and the electric cables on the other, and are well lighted. There is no portion of the wiring or piping underground which is not accessible at a moment's notice, and one may travel these subways in a "Sunday Suit" without harm to it.

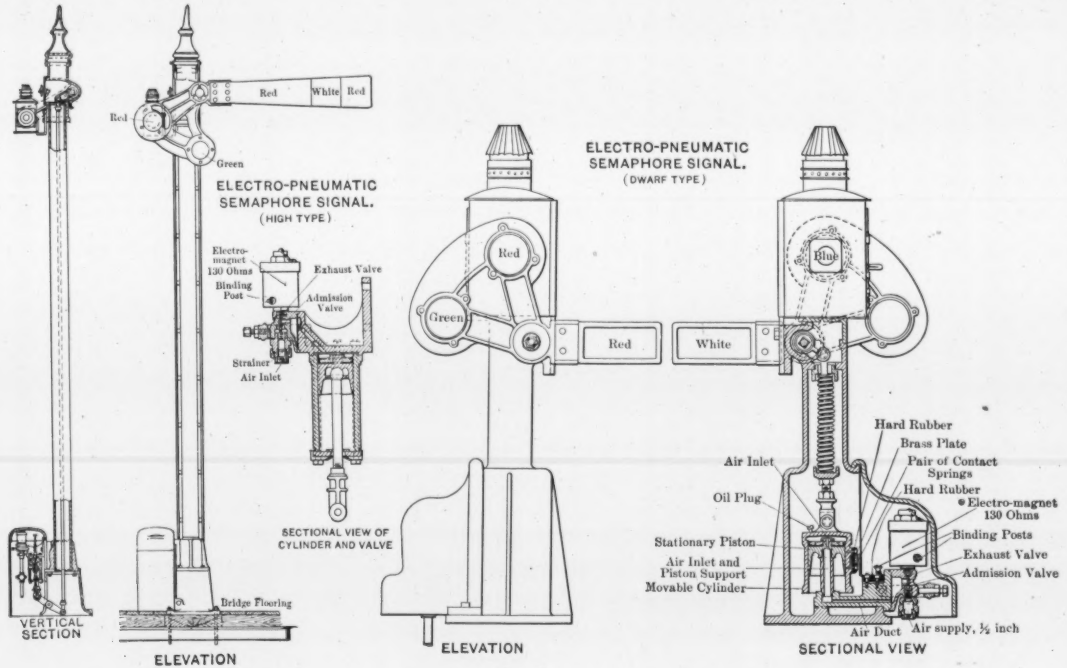

VERTICAL SECTION
Red
Red
White
Red
Green
ELECTRO-PNEUMATIC SEMAPHORE SIGNAL. (HIGH TYPE)
Electro-magnet 130 Ohms
Exhaust Valve
Binding Post
Admission Valve
Strainer
Air Inlet
SECTIONAL VIEW OF CYLINDER AND VALVE
Bridge Flooring
ELEVATION
ELECTRO-PNEUMATIC SEMAPHORE SIGNAL. (DWARF TYPE)
Red
Green
Red
White
Blue
ELEVATION
Hard Rubber
Brass Plate
Pair of Contact Springs
Hard Rubber
Air Inlet
Oil Plug
Electro-magnet 130 Ohms
Stationary Piston
Binding Posts
Air Inlet and Piston Support
Exhaust Valve
Movable Cylinder
Admission Valve
Air Duct
Air supply, ½ inch
SECTIONAL VIEW

FIG. 11.

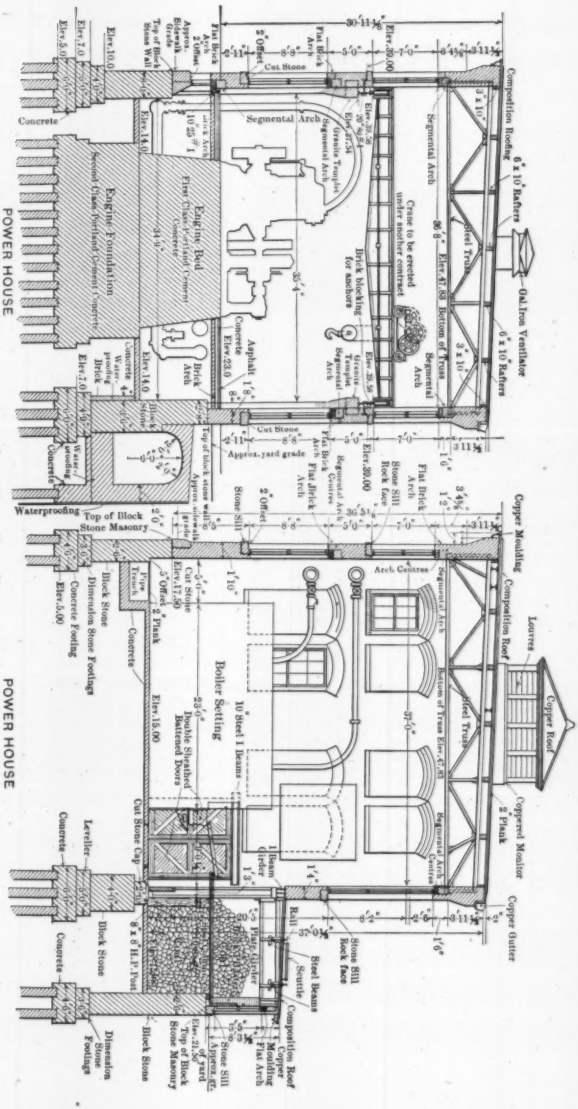

POWER HOUSE
SECTION THROUGH ENGINE ROOM
POWER HOUSE
SECTION THROUGH BOILER HOUSE
SCALE OF FEET
Engine Foundation
Second Class Portland Cement Concrete
Engine Bed
First Class Portland Cement Concrete
Segmental Arch
Crane to be erected under another contract
Brick blocking for anchors
Composition Roofing
Gal. Iron Ventilator
Steel Truss
Bottom of Truss
Cut Stone
Concrete
Waterproofing
Top of Block Stone Masonry
Approx. yard grade
Stone Sill
Rock face
Flat Brick Arch
Copper Moulding
Louvres
Copper Roof
Copper Gutter
Boiler Setting
10" Steel I Beams
Double Sheathed Battened Doors
Block Stone
Dimension Stone Footings
Concrete Footing
Leveller
Cut Stone Cap
Plate Girder
Steel Beams
Scuttle
Stone Sill
Rock face
Composition Roof
Copper Moulding
Flat Arch
Dimension Stone Footings
Block Stone
Top of Block Stone Masonry

FIG. 12.

Power House Equipment.

The buildings which contain this equipment are upon the Dorchester Avenue side of the yard, and in order that they might interfere as little as possible with room for tracks, they have been built tandem fashion; the ice plant nearest the head house, compressor and electrical generating plant next, and boiler house last. These buildings contain ice-making and refrigerating machinery, ice storage room, air compressors to make power for air brakes and the switches and signals, four Westinghouse direct-current generators, to make electric current for lighting, elevators and motors (with room for two more), a central heating plant, ten horizontal return-tubular boilers 72 ins. in diameter and 18 ft. long, an economizer, mechanical stokers, mechanical draught fans and engines, feed pumps, compressors and minor appurtenances.

The apparatus and the size of the units have been selected with special regard to the great variation in load required at different times of the day and year. There is, of course, considerable surplus power beyond absolute necessity, to provide for breakdowns and repairs. The steam piping has been duplicated in such a manner that any accident thereto will not seriously impair the operation of the plant.

The boilers are made from an extra quality of steel, and are designed to carry a working pressure of 150 lbs. per square inch. The shell plates are $\frac{1}{2}$ in. thick, and the heads $\frac{9}{16}$ in. thick, and are well stiffened by angle irons, and braced by five through bolts $1\frac{3}{4}$ ins. in diameter. Each boiler has 130 tubes 3 ins. in diameter, and is equipped with the usual manholes, blow-offs, safety valve, etc. They are suspended from channel beams resting upon wall plates carefully bedded upon the walls of the setting. The boilers are provided with an equipment of Roney mechanical stokers, and provision has been made for a 50% increase in the boiler plant.

The economizers are two in number, one of 520 pipes and one of 240 pipes. The settings are built for 1 040 pipes. The mechanical draught is provided by two slow-speed exhaust fans 14 ft. in diameter and 7 ft. breast, driven by two horizontal engines of sufficient size to drive the fans 100 revolutions per minute. Either or both fans are operated at will, and are designed to produce a draught through the steel stack 11 ft. in diameter, carried a few feet above the roof, equal to a 250-ft. chimney.

FIG. 1.—VIEW FROM CONGRESS STREET BRIDGE.

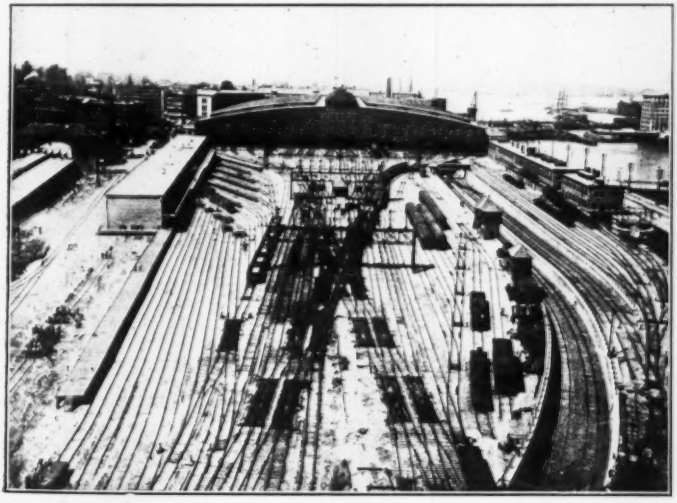


FIG. 2.—VIEW OF YARD AND BUILDINGS ABOUT JUNE 1ST, 1899.

The four Westinghouse, single-valve, compound, automatic engines have cylinders 18 and 30 ins. in diameter, and for 16 ins. stroke of piston, and are rated, when running at their standard speed of 250 revolutions per minute, at 375 I. H.-P.

They are operated non-condensing if the exhaust steam is required for heating purposes, or under a vacuum, as occasion requires. Each engine is directly connected to a 225 K.-W. Westinghouse multipolar engine-type generator, capable of generating current at two potentials, one of 220 volts for power, and one of 110 volts for lighting.

The switchboard is an interesting portion of the installation, but a technical description is out of place in this paper.

A traveling crane of 20 tons capacity spans the engine room.

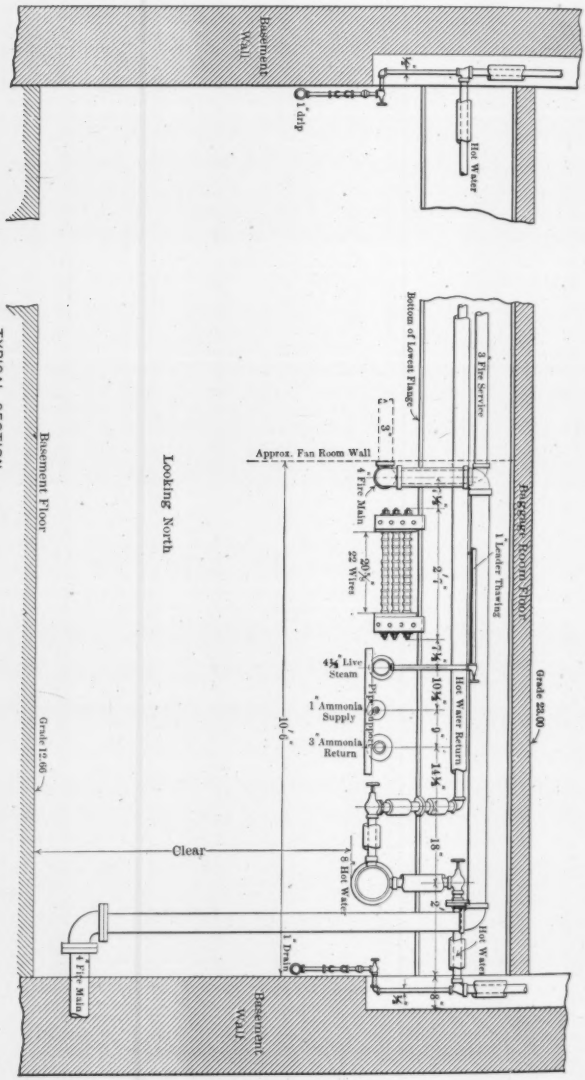
The other apparatus in the power house is described under the various headings which follow, in their appropriate connection.

The suction pipe from the condenser to salt water in the Fort Point Channel, is laid under Dorchester Avenue in a chamber which permits of entrance to adjust the flanged joints in case of air leaks, leaks being expected on account of settlement of the new street filling. The suction well and inlet chamber are built partially of timber and partially of masonry, and while serving the purpose properly, under the sea wall, have only the merit of being inexpensive.

Electric Lighting.

Before the amount of lighting for the various parts of the station was determined upon, a carefully prepared statement of the lighting of all the large stations in the United States was made, and from this as a guide the lights were arranged. The train-shed platforms, midway, baggage rooms, express platforms, express rooms, entrances and sidewalks are lighted by arc lights of enclosed pattern. In the train shed they are hung from the roof and steadied horizontally by the current wires and guy wires. The switches for the platform lights are located on the midway fence, and each platform may be lighted separately, if desired. The lights on the passenger platforms are 120 ft. apart. The suburban station is lighted by a large number of 5-light clusters of 16-candle-power lamps. These reflect against the enameled brick ceiling.

The waiting room is brilliantly illuminated by lights in the ceiling, and also by a row entirely around the sides.



TYPICAL SECTION
SHOWING PIPES AND WIRES
IN
DORCHESTER AVE. BASEMENT.
Fig. 13.

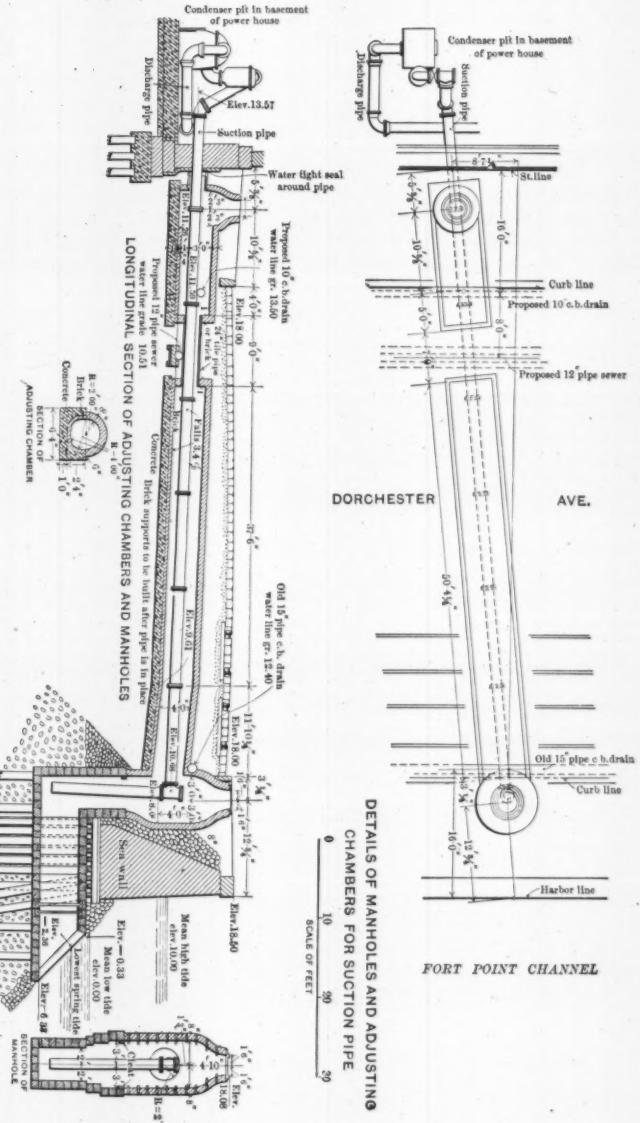

Condenser pit in basement of power house
Discharge pipe
Elev. 13.57
Suction pipe
Water tight seal around pipe
Proposed 10" c.b. drain water line gr. 13.50
Proposed 12" pipe sewer water line grade 10.51
Concrete
Brick supports to be built after pipe is in place
Old 15" pipe c.b. drain water line gr. 12.40
Sea wall
Mean high tide elev. 10.00
Mean low tide elev. 0.00
Lowest spring tide
Elev. 18.50
Elev. 18.00
Elev. 9.61
Elev. 10.08
Elev. 6.0
Elev. 2.36
Elev. 6.33
Cleat
Brick
Concrete
SECTION OF ADJUSTING CHAMBER
SECTION OF MANHOLE
LONGITUDINAL SECTION OF ADJUSTING CHAMBERS AND MANHOLES
Condenser pit in basement of power house
Discharge pipe
Suction pipe
St. line
Curb line
Proposed 10" c.b. drain
Proposed 12" pipe sewer
DORCHESTER AVE.
Old 15" pipe c.b. drain
Curb line
Harbor line
FORT POINT CHANNEL
DETAILS OF MANHOLES AND ADJUSTING CHAMBERS FOR SUCTION PIPE
SCALE OF FEET
0 10 20 30

FIG. 14.

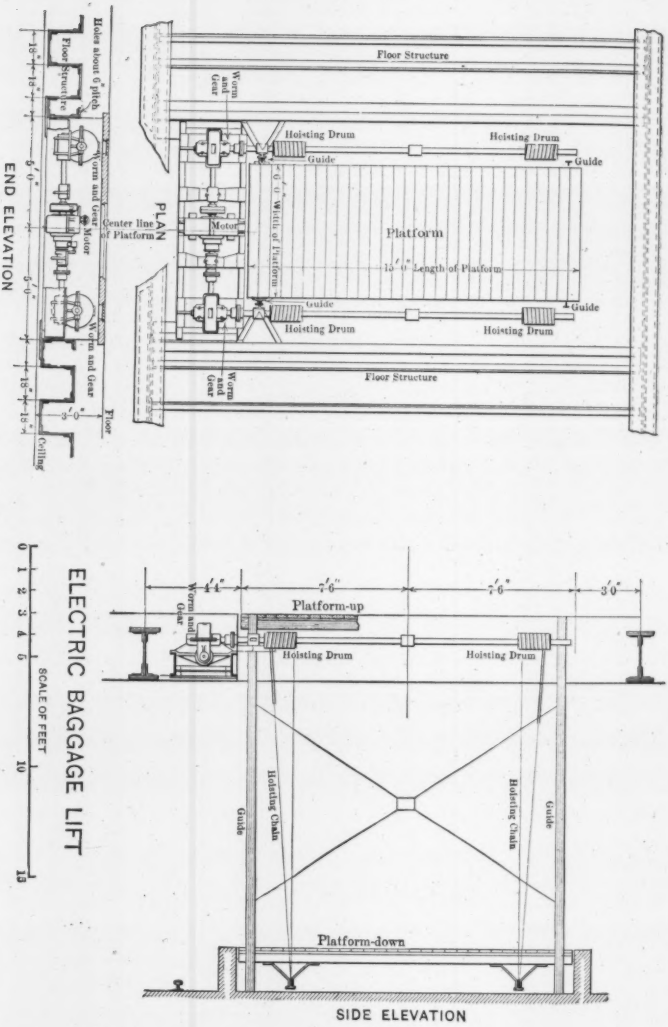

END ELEVATION
Holes about 6" pitch
Floor Structure
Worm and Gear
Motor
Center line of Platform
Floor
Ceiling
PLAN
Floor Structure
Worm and Gear
Hoisting Drum
Guide
Motor
Platform
15' 0" Length of Platform
6' 0" Width of Platform
Floor Structure
SIDE ELEVATION
4'4"
7'6"
3'0"
Platform-up
Platform-down
Hoisting Chain
ELECTRIC BAGGAGE LIFT
SCALE OF FEET
0 1 2 3 4 6 10 15

Fig. 15.

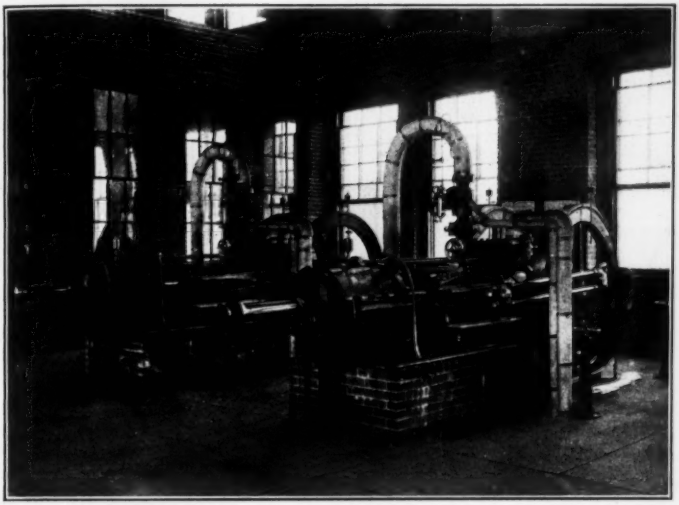


FIG. 1.—AIR COMPRESSORS.

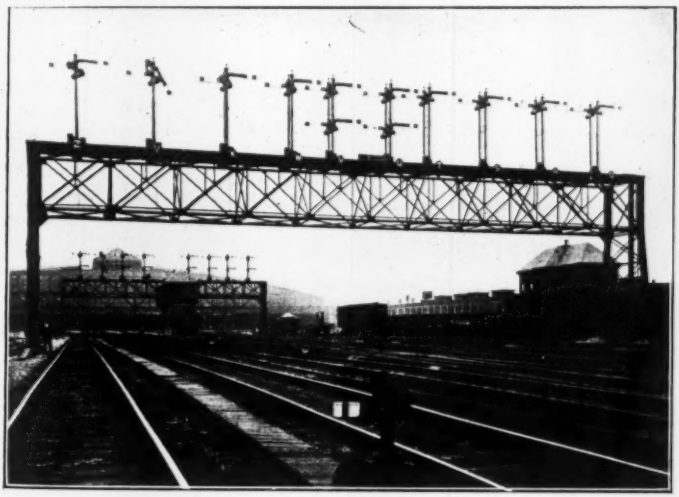


FIG. 2.—SIGNAL BRIDGE NO. 7.

The offices and corridors are lighted throughout with electricity. There are no illuminating-gas pipes in the building.

There are nearly 300 arc and 7 000 incandescent lights.

All the wires are carried in water-tight iron pipe ducts throughout the walls and floors of the building, and are accessible at the various drawing-in, cut-out or junction boxes. There are 18 sets of feeders running from the generator switchboard to the distributing centers about the premises, where there are switches for local control of the lights.

Elevators and Lifts.

All the elevators and baggage and express lifts are run by electricity. There are 5 public elevators and 2 private elevators in the head house, 1 ice lift in the ice plant, 8 baggage and express lifts in the train shed, 4 in the baggage rooms and 5 in the express buildings, making a total of 25. They were all, with the exception of the ice lift, constructed by the Sprague Elevator Company.

The passenger elevators are of the usual pattern, and have no special features.

The baggage and express lifts are of special pattern, and are one of the most interesting features of the mechanical plant. On account of the waterproofing sheet below, which could not be punctured, and the fact that most of these lifts were out of doors, it was not feasible to adopt hydraulic lifts. Further, on account of the impracticability of having any framework more than 3 or 4 ft. above the platforms, between the trains, the baggage platforms being restricted in width so that posts to such framework would be too near the car windows, it was not practicable to have overhead sheaves and ropes. Study, on the part of the contractors, brought out a type of electrical machinery which could be contained in the thickness of the floor. This consists mainly of a revolving shaft on each side of the elevator openings, near the top, which shafts wind up four chains, one near each corner of the lift platform, these shafts being rotated by worm gears, at right angles to them, at one end of the lift, the thrust of one worm gear reacting against the other, and the countershaft being rotated by an electric motor in the floor. This apparatus, being suitable for platforms between trains, and being simple and economical, was adopted for all but two (the two-story lifts in the express buildings), of the seventeen lifts.

The platforms of these lifts are 6 ft. wide and 15 ft. long, and the lifts are designed to carry with perfect safety 3 000 lbs.

The problem of making suitable protection around the opening in the platforms, and safety gates at the ends, was perplexing, but was solved by making a very strong sheet-iron frame for each side, about 3½ ft. high, and placing in each end a swinging gate. This framework cannot be wrecked by a heavy truck, and the gates cannot be struck, when open, by the trucks. The gates are connected, so that when one opens the other opens, and are also arranged by contact latches, so that the lifts cannot be moved if they are open, neither can the gates be opened if the lift is below the platform level. It is necessary to unlatch the gate by hand, which can be done from either end, before the gates are free to open. The baggage-room lifts are not protected by the same style of gate, but the principles are the same. The lifts are operated by foot-pushes at both ends of the platform, so that a man can work the lifts at whichever end he may be. A push on one button raises the lift, and on another button lowers the lift. They are all supplied with call bells and automatic gates at the lower level.

It was necessary to construct the platform over the motors, so that moisture could not find its way to them, and at the same time have the platforms removable, so that the motors and machinery could be got at. After a little study this was arranged without difficulty, and these lifts have given excellent satisfaction.

Heating and Ventilating.

The head house and public rooms are heated by a hot-water system. The water is heated in three tubular heaters, two of them connected to obtain heat by the circulation of exhaust steam from the engines, through them, and the third connected in a similar manner for live steam. The hot water is circulated through the tubular heaters and through a loop system of piping around the head house by means of two centrifugal pumps, one a relay to the other, these being located, along with the tubular heaters, in a room in the power house, and called the central heating plant. The main rooms and offices are warmed by hot air, the air being drawn through down-take shafts from the roof, over hot-water stacks, or radiators, in the basement, by large fans, and forced through one duct to each room, so heated, at a temperature of about 70° Fahr., and through another

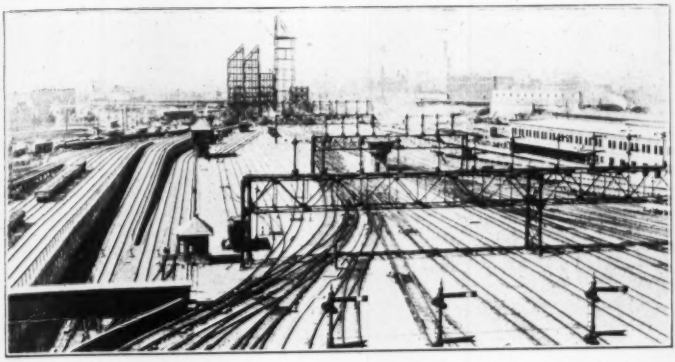


FIG. 1.—EASTERLY PORTION OF YARD AND SUBWAY.

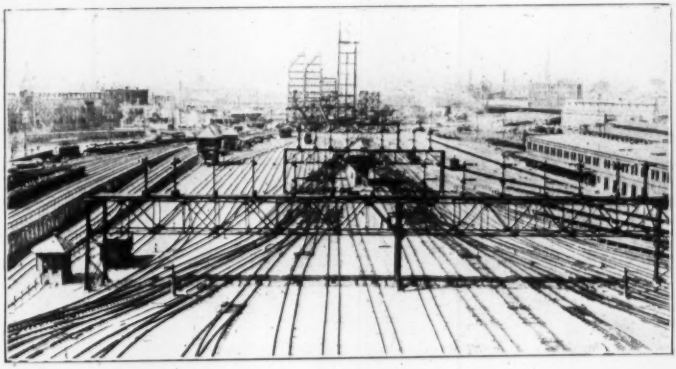


FIG. 2.—MIDDLE PORTION OF YARD.

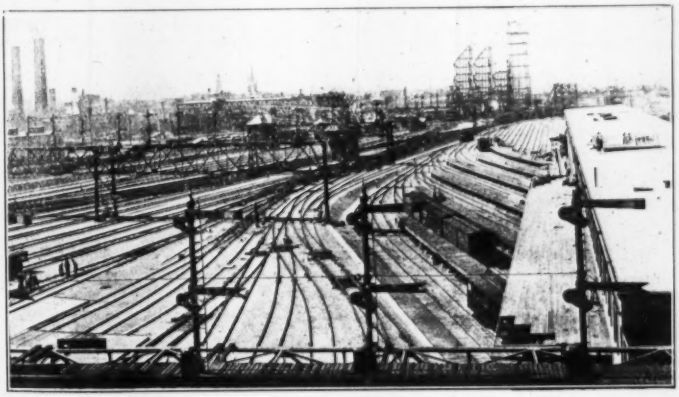


FIG. 3.—WESTERLY PORTION OF YARD.

duct, over a supplementary heating stack, to each room at a much higher temperature. At the register in each room is a mixing damper, so that the inmate can take a full volume of the air at a temperature of about 70°, or at a much higher temperature, at will.

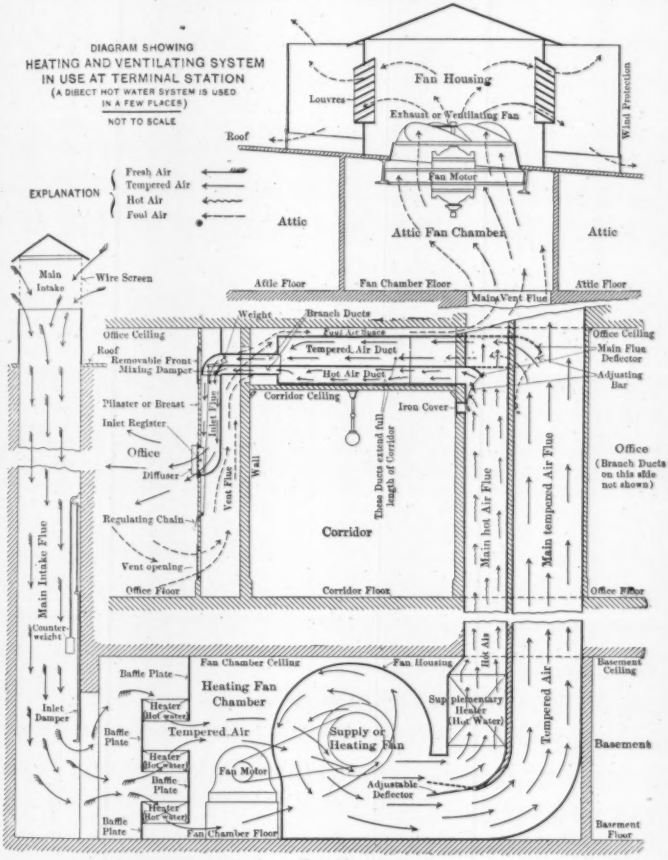


FIG. 16.

The main loop in the system is about five-eighths of a mile long, and large bends have been used freely to take care of expansion and contraction.

There are six fan rooms, of different sizes. In the summer these fans are run to assist in ventilating the offices.

The baggage rooms, some of the corridors, and other much exposed offices are heated by direct hot-water radiators. It is not expected that the baggage rooms will be kept much above freezing in cold weather.

The main ducts of the heating and ventilating system are over the ceiling of the corridors, and all hot-air ducts are thoroughly insulated. The hot-water pipes are all insulated. It is possible to circulate the water at different velocities and at different temperatures, and these are the means used to regulate the heat. There are 12 ventilating fans in the attics which exhaust the air from the office and toilet rooms through special ducts to the outlet chambers on the roof.

All the fans in the heating and ventilating system are driven by electric motors.

All toilet rooms are heated by the direct method, to prevent the accumulation of air pressure in the rooms, which would cause the escape of foul odors through the doors, and are ventilated by special exhaust fans at the roof, fresh air finding its way to these rooms through screened doors.

Ice-Making and Refrigeration.

An ice-making plant, capable of making 20 tons of "Diamond" ice per day, frozen from one side only, upon the plate system, has been installed in the power house. Beneath the freezing tanks an insulated storage vault, large enough to hold 800 tons of ice, has also been built. This ice is used in the cars, restaurant, and some of the offices, and is made from thoroughly filtered water, taken from the city mains, and harvested in large blocks, cut in the freezing tanks by traversing steam cutters, lifted out of the tanks by a traveling pneumatic lift, and lowered to the required level in the storage vault by an electric lift. Later on, it is weighed, cut up into small pieces, and delivered upon demand to the different users.

Two ammonia compressors are provided, one sufficient for the ice-making and refrigeration purposes, and the other for the latter alone.

A refrigerating plant, on the brine system, is installed for the purpose of cooling the storage boxes (of about 4 000 cu. ft. capacity) in the restaurant, and for cooling drinking water, which may be drawn from

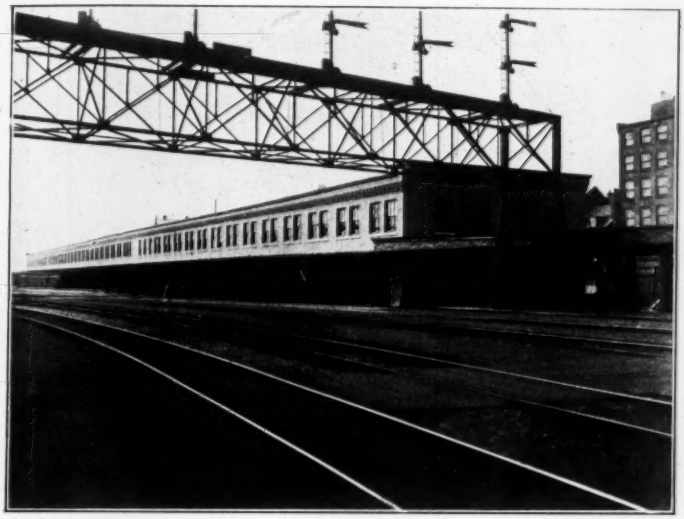


FIG. 1.—TRACK SIDE OF EXPRESS BUILDINGS.

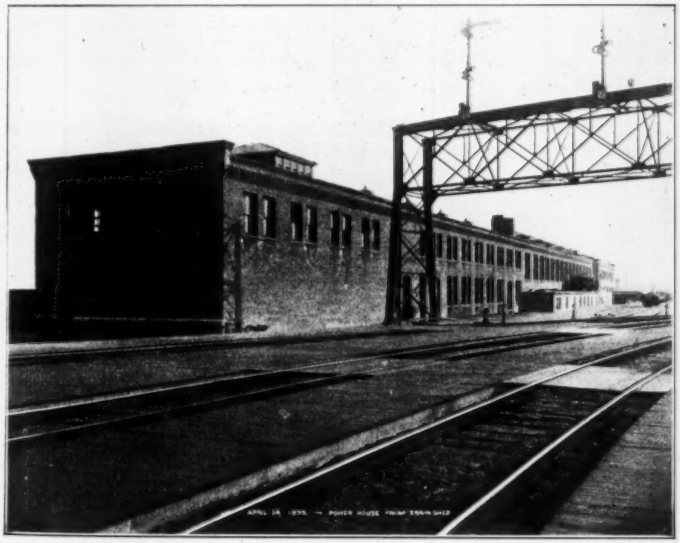


FIG. 2.—POWER HOUSE.

about 30 drinking fountains located in the corridors and public rooms at various places throughout the head house.

Steam and Hot Water for Head House.

A piping system for a supply of live steam and hot water has been installed in the head house for cooking and warming purposes in the kitchen and serving rooms, and for supply to outlets in the pay lavatories, outlets for cleaning purposes, and for the shower baths installed for the train men. All the piping is insulated, and the total number of outlets is about 35.

Car Heating.

At all the stub-track ends in the train shed and throughout the express yard, 47 places in all, provision has been made for furnishing steam heat to the cars. The steam pipe at the ends of the tracks in the train shed is on the loop system, and any break in the main pipe does not interrupt the supply to any great extent. Perhaps the only interesting feature in this provision is the style of metallic universal-joint connection to the car hose, which is what is known as the McLaughlin joint, a recently devised joint of very simple pattern, hardly to be described without detail drawings.

Air Brake Testing.

Two Westinghouse air-brake pumps are installed in the power house to supply air for testing brakes and charging the cars, and this air is delivered at the 47 stub tracks in the train-shed and express yards. A main and several auxiliary reservoir tanks are used to provide storage for excessive draught at times, and provision is made for draining off the condensation water at the various low points in the pipe lines, and at the reservoirs. At the track ends in the train shed the air passes through rubber hose to the car connection. Out of doors the air passes through the McLaughlin metallic universal joints. The latter feature is a new departure for air connections.

This system of compressed-air piping and machinery is kept separate from the compressor plant for switches and signals, so that a failure in the former will not affect the more important work of the latter. There is no physical reason why they could not be arranged as one plant, if so desired.

Fire Protection.

A 4-in. water-pipe connection has been made at five different points with the high-service mains of the city water-works, and fire-riser pipes, 3 ins. in diameter, with hose and nozzles, have been installed all over the premises. Altogether, there are 104 outlets for 2½-in. hose in the main building, and 2 outlets in the interlocking towers for 1½-in. hose, 6 of the former being on the roof of the main building. No outlets for this fire supply or any other hot or cold water supply are located over the main waiting room.

In addition to the above, there have been installed about the head house several auxiliary fire-alarm boxes, connected with the city fire-alarm system, which can be rung from 22 push buttons located in various places. Additional hose, on reels and in boxes, has been provided for emergency. No fire-fighting organization of employees has been made, reliance being placed on the city department, which can be had at immediate notice. One fire, which did no great damage, caused by a blacksmith's forge on the roof, within a few days after the station was opened, was observed in the attic by an employee, and extinguished with water from the established system, without recourse to the city department, thus illustrating the virtue of this private system.

Frost Protection.

The twenty-two 8-in. and the two 12-in. down-spouts from the train-shed and midway roofs are jacketed with non-conducting covering, and the jacket spaces are warmed in freezing weather by coils of 1-in. steam pipe. The jacketed space is 1½ ins. wide, and wire lathing serves as a base for the non-conducting covering. This covering is an asbestos compound, and it is protected by heavy canvas, well painted. The live steam for the coil, which is two runs of 1-in. pipe, forming a loop by connection near the roof, is supplied from a distributing system, having its origin in the car-heating main.

These jackets have served to keep the down-spouts open one winter, and it is hoped that they will prove as valuable in the future. A 2-in. brass pipe to carry live steam has been installed entirely around the midway roof in the gutter, to keep the gutter open. No other steam pipes have been provided for melting snow.

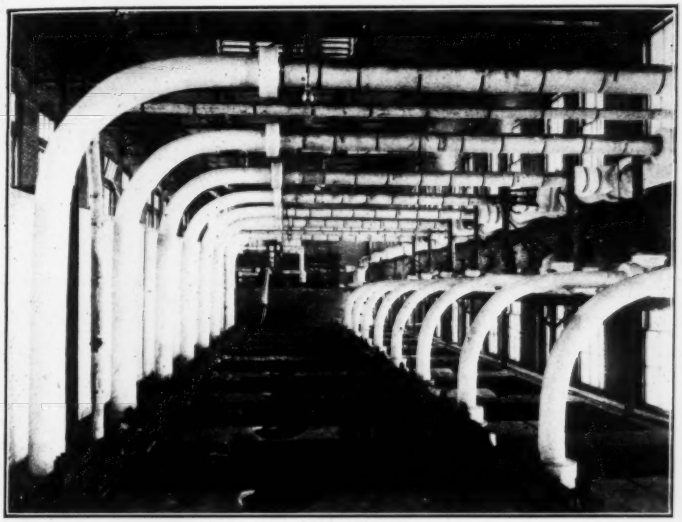


FIG. 1.—PIPING, OVER BOILERS.

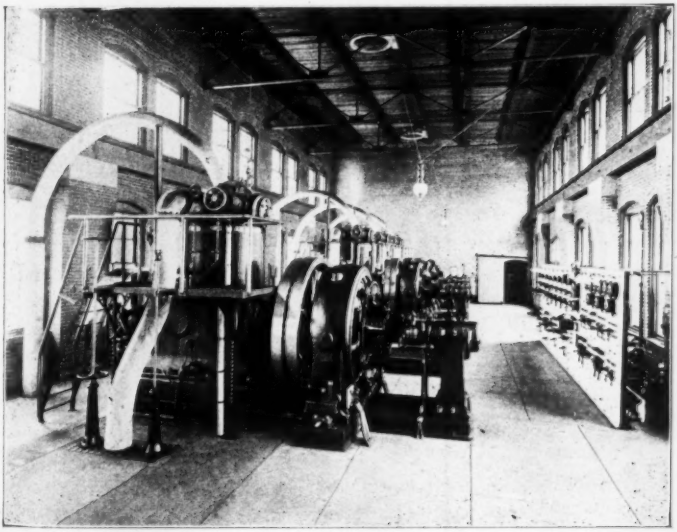


FIG. 2.—INTERIOR OF ENGINE ROOM.

Pumping Plant.

As the entire suburban station is below the level of the ocean, and as part of the track-approaches to it are open to rain and snow, it has been necessary to provide a pumping plant in a sump well, located near the power plant, to take care of storm water and any seepage water which may eventually get through the waterproofing sheet. This plant consists of two electrically driven, vertical-shaft, centrifugal pumps. One has a capacity of 4 000 galls. per minute, and the other 1 000 galls. per minute, against a head of 16 ft. The smaller pump is arranged to work automatically, being governed by a float. The other is started by hand upon an electric alarm signal controlled by a float, giving notice in the power house.

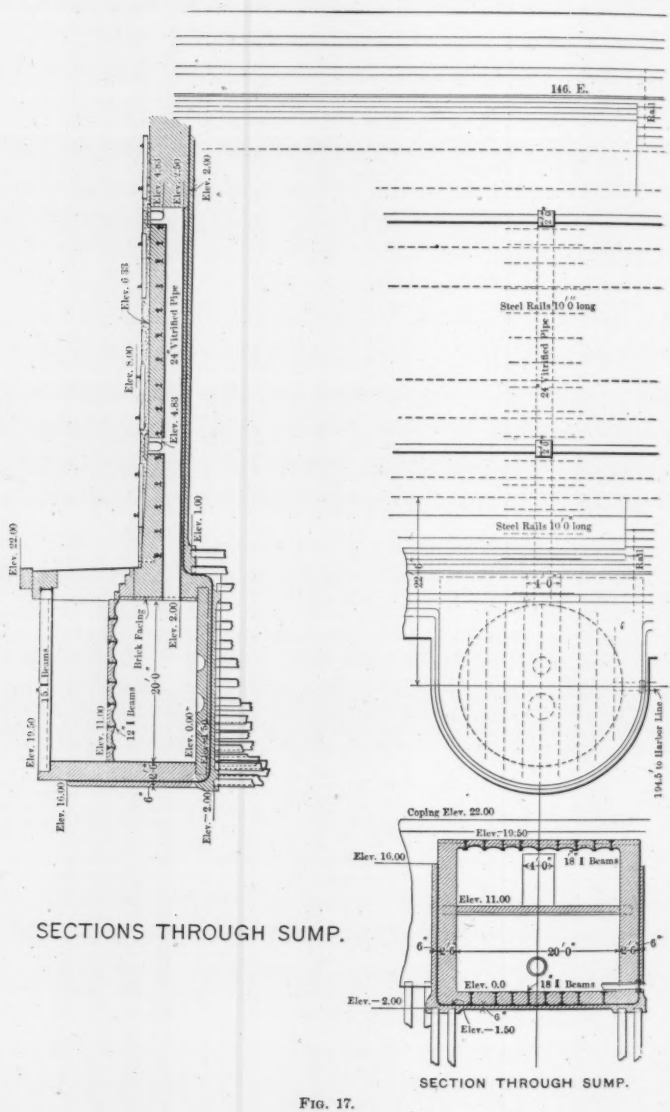
These pumps discharge through check valves to a pipe connection with the city storm sewer, which crosses the premises and empties into Fort Point Channel.

Thus far, the pumps have seen very little service, and it is to be hoped that it will not be increased.

At extreme low tide the small accumulation of water during the high-tide period finds its way through the check valves, which act as automatic flood gates to empty the sump well without the pumps. If the check valves fail to work on the rising of the tide, the alarm rings in the power house, and the emergency stop valve in the discharge pipe is immediately closed before any damage can be done.

Coal Handling.

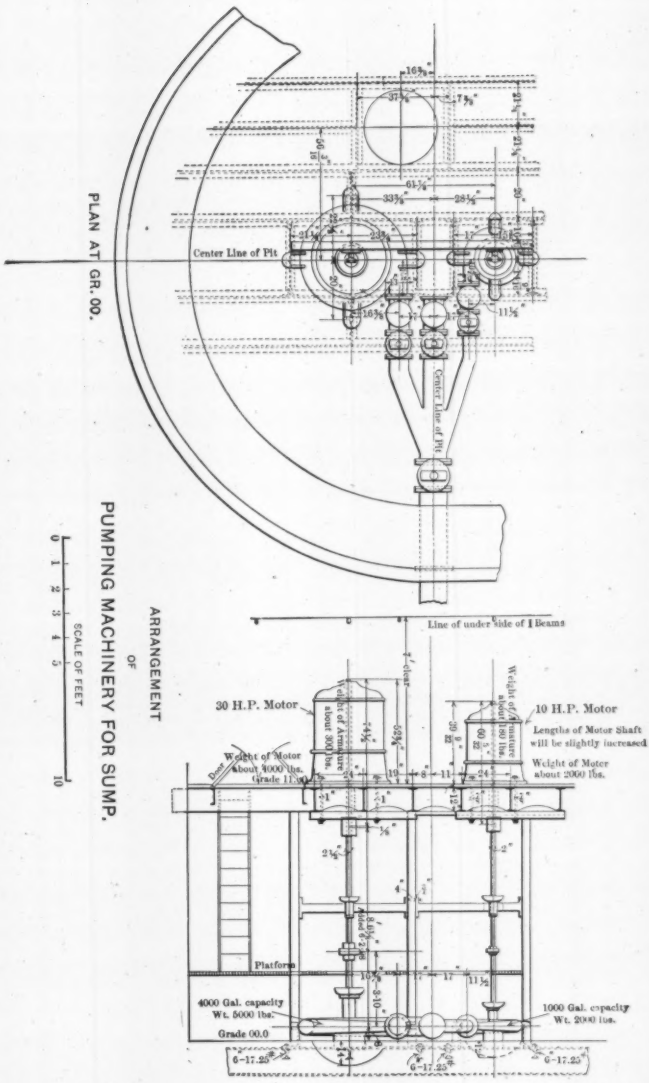
As the Terminal Company has no water front suitable for the discharge of coal from vessels, the coal is brought from one of the coal pockets belonging to one of the operating railroad companies, on cars which permit of unloading through hoppers in the floor. These are set over a coal vault, constructed in the rear of the boiler house, as shown on the cross-section plan, Fig. 12, and the coal is then dumped upon an intermediate floor, level with the hoppers of the mechanical stokers, this floor having storage capacity for several days' supply. From this floor the coal is pushed or shoveled into the mechanical stokers. Beneath this intermediate floor is further storage room for several days' supply, which supply is held in reserve against snow blockades in the winter, when it would be difficult to move the cars. From this lower floor it is necessary to lift the coal about 5 ft. to put



SECTIONS THROUGH SUMP.

SECTION THROUGH SUMP.

FIG. 17.



ARRANGEMENT OF PUMPING MACHINERY FOR SUMP.

FIG. 18.

it into the stoker hoppers. Originally, it was not intended to have mechanical stokers, and to fire by hand from this lower level without this 5-ft. lift; but, after the stokers were installed, the intermediate floor was arranged, and the handling simplified, as well as the storage room increased.

Storage capacity for several thousand tons was considered, but on account of the relatively small consumption, varying with the seasons, from about 8 tons to 35 tons per day, the difficulty of placing the storage close to the boilers, the necessity of considerable coal-handling machinery, if not so placed, the necessity of using cars in any event, and the ability to take advantage of low prices, due to bulk deliveries, by purchasing in connection with one of the operating railroad companies, such storage facilities were not constructed.

At present, the ashes are wheeled from the furnaces to an area just outside of the boiler room, where they are taken away daily by private parties without cost to the Terminal Company. They are eventually disposed of either for cinder-concrete material, or to go to the South Boston flats for wharf filling, where a price is paid sufficient to cover the cost of handling.

Cost of Power Plant.

The cost of the entire power plant, including all piping, wiring, electric lighting of every sort, fans, heaters, radiators, air fixtures, frost jackets, ventilating system, steam connections, refrigeration, hot-water pipes, lifts and elevators in the head house, subway, express and gas buildings and train shed, as well as pumping plant in the sump well, has been, not including the interlocking, upwards of $400 000. The various parts are so inter-related to each other that no attempt will be made in this paper to give unit costs for elevators, lifts, lights, etc.

Pintsch Gas.

A plant for the manufacture of Pintsch gas for use in cars has been established on the terminal grounds, capable of making 120 000 cu. ft. of gas daily.

A building, 110 ft. long and 40 ft. wide, two stories high, of similar type to the power house, has been constructed, on pile foundations; and in this building there have been installed 40 retorts and the accompanying furnaces, 3 engines and compressors for compressing the

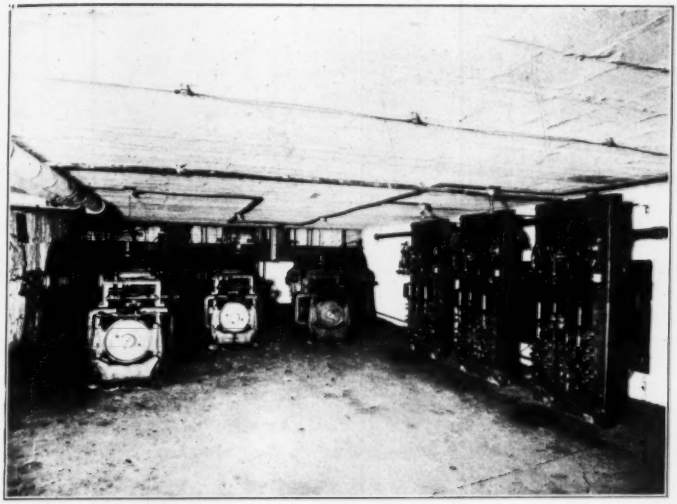


FIG. 1.—APPARATUS FOR THREE MAIN ELEVATORS AND CONTROLS.

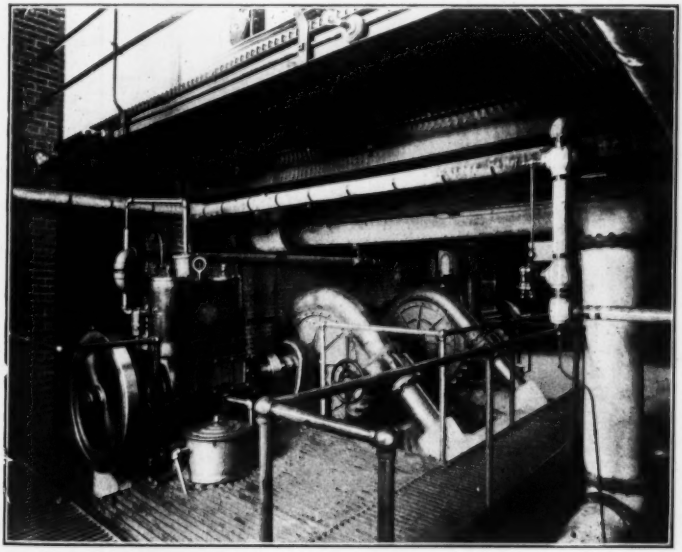


FIG. 2.—CIRCULATING PUMPS, CENTRAL HEATING PLANT.

gas, purifiers, meter, feed tanks, drip and tar pumps, and all other necessary appliances. The building is lighted by electricity. Outside of the building are installed 2 oil-storage tanks, a tar tank, 10 store holders and a gas holder 30 ft. in diameter and 15 ft. deep. A main pipe leads from this plant, connecting with distribution pipes to 120 outlets in the train shed and about 20 more outlets in the express yard, where gas can be put into the car tanks through hose connections. The cost of this entire gas plant, including the building, is about $80 000.

Material Used.

The material used in the work approximates: 43 000 spruce piles, 15 100 000 common brick, 487 000 medium brick, 846 000 enameled brick, 74 000 cu. yds. concrete, 32 000 cu. yds. stone masonry, 30 000 000 lbs., or 15 000 tons steel, equal to about 1 200 car loads; 200 000 cu. ft. of cut stone for building, or 500 car loads; 75 000 bbls. Portland cement, 20 000 bbls. Rosendale cement, 8 000 bbls. coal-tar pitch, 6 500 bbls. prepared asphalt, 850 000 lbs. tarred paper, 450 000 lbs. sheet copper for roof trimmings; 5 000 000 ft., B. M., yellow pine timber, for various uses; 16 000 lbs. solder, 10 acres gravel roofing, 150 000 sq. ft. wire-glass and 40 000 lbs., or 20 tons, of putty to set the same.

Costs of Yard Buildings, Per Cubic Foot.

The power house, including foundations	$0.121
The gas building, including foundations	0.135
The express buildings, including foundations	0.131

The total cost of these buildings aggregates $310 000.

These buildings are of brick, two stories high, with good basements, founded on piles, with iron floor-beams, iron roof-trusses, wooden floors and roof boards, roofs covered with tar and gravel, plumbed, heated and lighted, and the express buildings contain offices for 250 persons; also 5 truck lifts. The above costs do not include any foundations for machinery located within these buildings.

Cost of Head House.

The approximate cost of the head house, including foundations, heating, lighting and elevators, is $1 565 000, or $0.214 per cubic foot. In calculating the cubic feet, outside dimensions and height from floor of basement to top of roof are used.

Cost of Train Shed.

Area, 343 140 sq. ft. Weight of steel, 23 lbs. per square foot. Total cost, including foundations, steel and covering, $382 180. Cost per square foot, horizontal area, $1.11. If the area and cost of the midway and connecting roofs is added to the train shed, the cost is $1.05 per square foot, horizontal area.

Organization for Construction.

The Terminal Company is managed by a board of trustees, one for each of the five constituent railroad companies.

The members of the board were as follows:

Charles P. Clark, Chairman, representing The N. Y., N. H. & H. R. R. Co.
Samuel Hoar, Vice-Chairman, representing The B. & A. R. R. Co.
Charles L. Lovering, representing O. C. R. R. Co.
Royal C. Taft, representing B. & P. R. R. Corporation.
Francis L. Higginson, and later, John M. Hall, } representing The N. E. R. R. Co.

The construction work was carried out under the direction of the architects, Messrs. Shepley, Rutan and Coolidge, and the Resident Engineer, the former planning and supervising the brick and stone head house and office-building construction, and the latter the foundations, train shed, midway, tracks, subways, express buildings, power house, sea wall, etc., and supervising the construction of the power plant and interlocking, each of the above parties reporting directly to the Board of Trustees.

The Chief Engineers of the operating railroads and the Manager of the Terminal Company exercised general criticism, and were consulted in regard to the requirements. All the large contracts for construction were executed on behalf of the Terminal Company by a member of the Board of Trustees. Other contracts were executed, under authority of the Trustees, by the Architects or Resident Engineer.

The general plans for the steel construction in the train shed and bridge floors, prepared by the Resident Engineer, were developed by J. R. Worcester, M. Am. Soc. C. E., of Boston. The shop cards were drawn at the office of the contractors, the Pennsylvania Steel Company, J. V. W. Reynders, M. Am. Soc. C. E., Superintendent, and were examined, checked and approved by the engineering department of the Terminal Company.

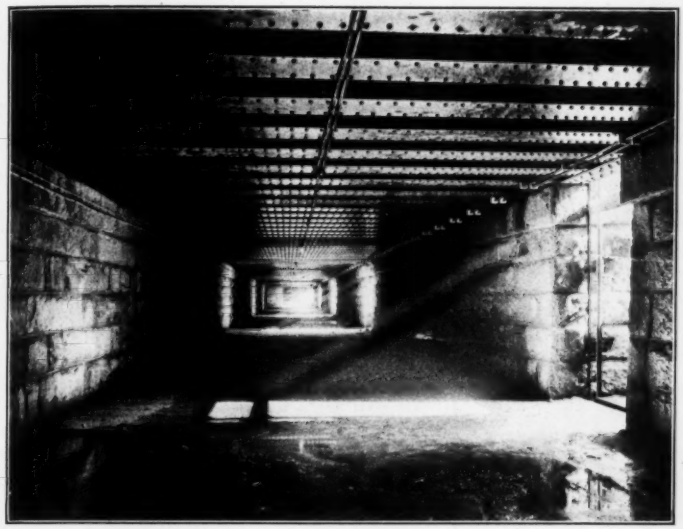


FIG. 1.—EXPRESS SUBWAY.

FIG. 2.—TRAIN SHED, SHOWING LEADERS ENCASED IN FROST-PROOF JACKETS.

The interlocking plans and details were made by the Union Switch and Signal Company, of Swissvale, Pa. The power plant, including lighting, heating, ventilating, elevators, lifts and ice-making machinery, was designed by or under the direction of Westinghouse, Church, Kerr & Co., of New York; Henry J. Conant, their Boston representative, the Sprague Elevator Company, and Professor S. Homer Woodbridge, heating and ventilating engineer, contributing to this end. The plans for the Pintsch gas apparatus were designed by the Pintsch Compressing Company, of New York.

The engineering department of the Terminal Company, assisting the Resident Engineer, embraced A. B. Corthell, M. Am. Soc. C. E., First Assistant Engineer; W. F. Goodrich, M. Am. Soc. C. E., Assistant Engineer, in charge in the field; J. H. O'Brien, architectural draughtsman, as well as other draughtsmen, instrument men, inspectors and helpers, all of whom, as well as the head men for the various contractors, faithfully carried out all the work entrusted to them.

Organization for Operation.

The organization for operating the station is at present substantially as follows:

A Manager, to whom all reports are made, and who himself reports to the Trustees.

For the Transportation Department, a Station Master and an Assistant Station Master for day and night service, with the necessary force of guides, gatemen, porters and cleaners.

A Ticket Agent, with necessary force, Yard Master and Assistant Yard Master, Baggage Agent and Parcel Agent, each with necessary force.

For the Maintenance Department, the following heads, reporting upon all matters pertaining to construction, repairs and operation, excepting operation of the interlocking plant, through the Resident Engineer to the Manager: Superintendent of Power Plant, including all matters pertaining to mechanical and electrical equipment, lighting, heating, ventilation, elevators, ice and gas machinery, with the necessary force; Supervisor of Buildings, with necessary force; Track Master, with necessary force. There is also a Supervisor in charge of all apparatus and employees supplying steam, compressed air and Pintsch gas to cars, he reporting to the Manager.

The Treasurer of the company, who at present is also Acting

Auditor, Pay Master and Purchasing Agent, reports directly to the Board of Trustees, the same as during construction.

Train Service.

The natural position of the four roads entering the station will be maintained in the train shed, as far as possible. There being eight main tracks through the throat, two can be devoted, one in and one out, for each road; and only the necessary crossing of trains of one road over the tracks used by another road, done, as is necessary, to reach baggage rooms, express yards, or other special service. As two-thirds of the 737 trains using the station are suburban trains, without baggage or express matter in quantity, this very much reduces interference upon the grand crossing. Those trains carrying large quantities of baggage and express matter must, of course, get as near the inward or outward baggage rooms as possible. It has been found thus far, in operation, that the baggage lifts expedite rather than hinder the free movement of baggage, as the free cross-run of the trucks beneath the tracks, compared with dodging trains on the surface, more than compensates for the time required to use the lift.

Fencing.

The property has been surrounded as far as possible on its boundary limits with a fence 8 ft. high; a part of this, adjacent to the streets, being a tight board fence, and upon division lines a picket fence, with proper gates for entrance and exit to the express and mail yards The subway and sea walls are lined with a heavy two-pipe rail fence with cast-iron posts.

Minor Conveniences.

It is impracticable in this paper to describe the large number of minor conveniences which have been arranged for this station. The mention of a few of them will suggest their importance and help to complete the record.

A private telephone exchange for the terminal only.

Separate ticket-selling booths for each man.

A liberal number of weighing scales in baggage rooms and express buildings.

Ample bicycle racks, check racks, etc.

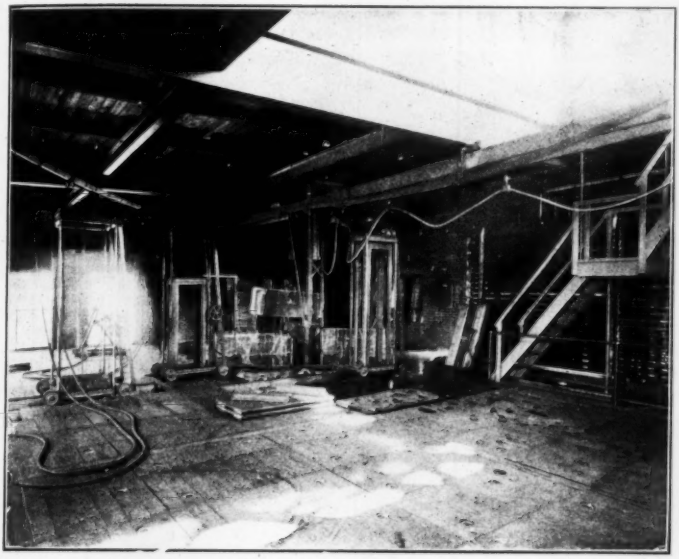


FIG. 1.—VIEW OF ICE TANK.

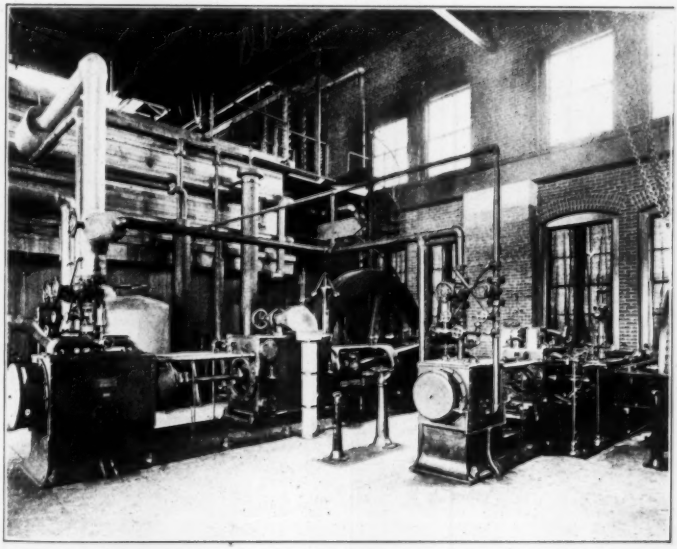


FIG. 2.—VIEW OF AMMONIA COMPRESSORS, ICE PLANT, ETC.

Checking system to care for trainmen's coats, hats, etc., instead of the usual lockers.

Kitchen and restaurant furnishings.

Booths for weighers at scales in express buildings.

Apartments for car inspectors.

Train mail-chute and sorting room.

Speaking tubes.

Case for sale of emergency articles, rubbers, umbrellas, etc., in women's room.

Shoe-cleaning and polishing chair in women's room.

Room for police squad at the station.

Dressing room for porters and gatemen.

Conveniences for handling newspapers.

Carpenter and paint shop for repair men.

Blacksmith shop for interlocking and track work.

Shop for small repairs to machinery and piping.

Stock rooms, oil house, lamp room, trackmen's tool room, yardmaster's office, trackmaster's headquarters and interlocking supervisor's room, storage yard for spare track material, etc., etc., etc.

Conclusion.

In conclusion, it gives no little satisfaction to the engineering department to say that throughout the entire construction there has been absolutely no friction between any of the parties who have contributed to the grand result, either as railroad officers, managers, architects, engineers, contractors and their representatives, or city and state heads of departments and officers, and that uniform courtesy has prevailed among all.

While it is not always the duty of the engineer to discuss the wisdom or necessity of certain lines of policy, and while such discussion is more or less unprofitable, it may, nevertheless, in this instance, prove valuable to answer in advance the questions sure to be raised as to whether such a large terminal, with its accompanying expenditures, is a proper investment for a railroad company to make.

When railroad stations have been utterly outgrown, as was the case in Boston, and the traffic cannot be properly handled, there is "no alternative" for the railroad management but reconstruction. The question whether each of the existing stations shall be enlarged, per-

haps under great difficulties, or whether a combination shall be effected, as in Boston, for the mutual convenience of the transportation companies and the public, is usually well weighed and decided by those most competent to decide. Having decided to build a new station, no management can be criticised for making it an up-to-date and even ahead-of-date structure.

Of the great expenditure necessary for the new terminal station in question, probably nearly two-thirds of its entire cost for land and buildings reverts directly or indirectly to the railroads interested, in lands released at old stations for other purposes or for entire disposal. With bonds selling at a premium on a $3\frac{1}{2}\%$ basis, much more can be accomplished, with the same interest charge, than was possible a few years ago, with interest at 6 per cent.

As far as can now be determined, the cost of operating the new and much more commodious station will be about the same as the sum of operating the old stations.

A railroad station is a minor appurtenance to the road itself, and it is impossible to always incur expense for renewals, on the basis of an investment upon which a new direct return will be forthcoming.

It may be compared, roughly, to a new storehouse for a cotton mill. It is a necessary convenience for the handling of the business, but the mill (or the railroad) is the real earning factor.

The question has been asked: "How much money did the City of Boston contribute toward the construction of the station?" In reply, it may be said that not one dollar was contributed, directly or indirectly, by the city for land or structures upon the terminal location. The city, through its own departments, was authorized by the Legislature, to expend a sum of money for streets and bridge construction, outside of the terminal property, not to exceed $2 000 000.

At different times a great many suggestions have been contributed to the terminal management, regarding construction of train shed, arrangement of tracks, handling of baggage, operation of station, etc., etc. Many of these were ingenious, some utterly useless. As a rule, nothing is accomplished if we wait until perfection is reached, and it is undoubtedly better to go ahead imperfectly than not at all.

The station is now being operated without delay to trains or people, and the results aimed at have been reasonably attained. Later on, when the subway station is put into service, there should be still greater satisfaction in the general operation.

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DISCUSSION.

Mr. Conrow. HERMAN CONROW, Jun. Am. Soc. C. E. (by letter).—The method used by Mr. Francis in water-proofing the masonry throws new light on a subject upon which comparatively little has been written.

In the summer of 1896 the writer had charge of the construction of a water-proof subway, beneath the four tracks of a railroad in Massachusetts. The subway was about 100 ft. long and 8 ft. in clear width, with the floor about 5 ft. below the level of the water in a nearby creek. It was necessary, therefore, to build a masonry structure which would be impervious to water under pressure. The design of the subway called for a foundation of concrete, brick side and end-walls, and a brick-arch roof with concrete backing. The water-proofing materials were refined bitumen asphalt of the best quality and heavy tarred paper. Inside, the finished subway was planned to have a mosaic floor and sides and arched ceiling of glazed bricks; but these embellishments have nothing to do with the water-proofing, and were added merely as a veneer to the main walls of the structure. The subway was built in a soil of fine sand so full of water that it was not far removed from a quicksand.

In order to simplify the description, the methods used in water-proofing the floor, side-walls and roof are separated.

1. The floor was water-proofed as follows: A foundation course of concrete, 6 ins. thick, was laid upon the sand, and the top of this was roughly leveled and smoothed by filling the depressions with mortar. After this mortar had set, heavy tarred paper was laid to break joints, and over the paper was poured a layer of molten asphalt. (The thickness of this pouring will depend upon the temperature of the asphalt. Very hot asphalt will run out to a thickness of less than $\frac{1}{4}$ in.; but it can be used so that the pourings will be nearly $\frac{1}{2}$ in. thick. The thin pourings, however, give the best results.) On this asphalt, after it had cooled, another layer of tarred paper was placed, which was followed by another coating of asphalt. This process was continued until a water-proof covering about 1 in. thick was formed. This was composed of three layers of paper and three pourings of asphalt. This floor covering could have been made entirely of asphalt, but the tarred paper greatly increased its strength.

Upon this water-proof floor 6 ins. of concrete were next laid, except where a space was left for the asphalt in the vertical walls to join directly to the asphalt floor. The purpose of this concrete was to keep the asphalt permanently cool and also to withstand the water pressure from beneath. Steel rails, 2 ft. apart, were placed in the concrete, and the ends of the rails were bedded under the inside walls,

thus giving strength to the concrete, in addition to its weight, to resist the upward pressure, otherwise a heavier layer of concrete would have been necessary. Mr. Conrow.

2. To water-proof the side-walls of the subway the general scheme was to build a core-wall of asphalt, 2 ins. thick, between two brick walls. The greatest care was necessary, in making the junction at the bottom, where the asphalt core-wall met the asphalt floor, to secure a water-tight joint. The inside wall surrounding the core was first built

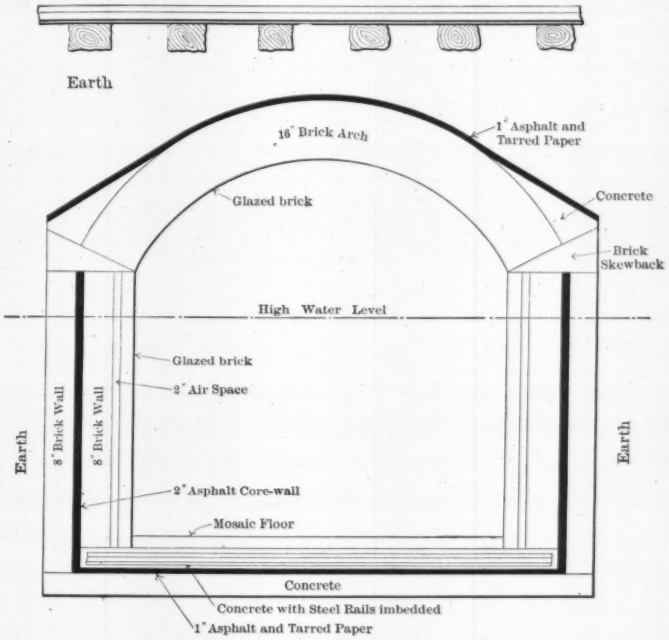


FIG. 19.

to a height of 6 ins. and the outside wall to a height of 1 ft., thus forming a narrow groove 12 ins. deep, into which molten asphalt was poured. In order to reduce the amount of asphalt, broken pieces of clean and dry brick were laid in the groove, which was then grouted full with very hot asphalt. The core-wall having thus been raised level with the brick walls about it, the brick walls were built up 12 ins. higher, and the process of asphalting repeated. In this manner the

Mr. Conrow. sides and ends of the subway were built to the desired height. When this had been done the walls and bottom formed a water-tight masonry box.

When starting the core-wall at the bottom, the asphalt was poured very slowly, for there was danger that the hot asphalt would melt that under the concrete and cause it to crack; but after the core-wall had reached a height of 1 or 2 ft., all anxiety on that score was dismissed.

To make a tight junction between one pouring of asphalt and another, the surface of the cold asphalt must be perfectly clean and dry. A film, either of dust or moisture, will prevent a water-tight junction. Sometimes, when the brick walls had been freshly built up above the core-wall, particles of mortar and drops of water collected upon the surface of the cold asphalt in the groove, and, in spite of brushing and wiping, it could not be cleaned and dried perfectly. In such cases a small quantity of kerosene was sprinkled on the surface of the asphalt in the groove and then lighted. The heat dried out the moisture and melted the surface of the asphalt, giving a perfectly new surface and making sure the unity of the work.

This use of kerosene the writer has found very valuable. Old surfaces of asphalt which had become full indentations and which were covered with sand which had become ground in, were rendered bright and clean by burning kerosene on the surface; the foreign particles disappeared by sinking deeper into the asphalt, leaving the surface in good condition to unite with the next pouring.

3. The water-proofing of the arch covering was very simple. After the masonry had been smoothed over with cement and it had set, asphalt was poured evenly over the surface, and, after cooling, a layer of tarred paper was laid, beginning at the sides and parallel with the barrel of the arch, working upward toward the crown, each course lapping over the previous one as in a shingle roof. Four layers of asphalt and three of paper were used.

The water-proofing of the subway by these methods was entirely successful, and upon the completion of the work the pumps were stopped and the water rose to a height of 4 ft. around the walls of the subway, which remained perfectly dry inside. Numerous heavy rains caused no leakage in the arched roof.

This method of water-proofing the side walls was not that originally tried upon this work. At first the attempt was made to water-proof a brick wall by means of paper and asphalt. After the wall had become dry it was coated with asphalt, and then paper was put on with lap joints. Another coat of asphalt followed, then more paper, and so on. A 4-in. brick wall was built adjacent to the paper to hold it and the asphalt in place. The results were not successful, and perfect water-proofing was not secured in this way. The writer is of the opinion

that tarred paper has little value as a water-proofing material for vertical walls which are to stand water under pressure, because of the impossibility of keeping the paper in its proper place. He also believes that if it is desired to build masonry absolutely water-tight the best way to do it is by using an asphalt core-wall where the asphalt may be poured into a groove and allowed to run into every crevice which will admit water. Mr. Conrow.

Both the asphalt and the tarred paper should be of the best quality. Cheap asphalt and the tar compound are not permanent substances, but will rot and become like powder under the action of water and air, thereby loosing all water-proofing qualities; and the poorer kinds of tarred paper are easily torn and are of little value. It will be found very difficult to handle the asphalt in hot weather since it becomes soft in ordinary summer temperatures. Cold asphalt is a durable and tough substance, capable of bearing considerable pressure, but if unprotected from the sun's rays, it becomes an unreliable material. If it is desired to use a large quantity of asphalt, the inconvenience of melting it quickly may become very great, especially if the asphalting outfit consists of but one or two kettles. One kettle will melt a large amount, however, if it is kept constantly full, since the molten asphalt in the kettle will quickly melt large blocks of the cold asphalt.

The accompanying section, Fig. 19, will aid in the illustration of the methods described.

J. R. Worcester, M. Am. Soc. C. E. (by letter).—The subject embraced by the paper is so comprehensive that the author has, of necessity, hastened over many features of the problem, a more extended account of which might have been interesting to the members of the Society. In the hope of shedding a little light on some of these points, the writer wishes to discuss briefly the portion of the paper which refers particularly to the train shed. Mr. Worcester.

Cantilever Principle.—To make more clear the reasons for adopting the cantilever style of trussing, it should be stated that before the method of supporting the roof was determined, the cross-section, so far as the roof line was concerned, was agreed upon almost exactly as it was finally built. This line was arranged so as to give a large vertical space for windows below the eaves and above the connecting roof, to enclose the whole width of the shed under a single roof and to avoid raising any portion so high as to overtop the head house. It was also determined that there should be two lines of intermediate columns placed substantially in the positions finally adopted.

With these conditions prescribed and with the manifest advantages of raising the bottom chord as much as possible, the cantilever principle naturally suggested itself as desirable. To ascertain whether it would result in any great economy, however, an estimate of the cost of trusses with the same outline, but broken over the interior columns,

Mr. Worcester. was made. It was found that the weight of trusses constructed on these lines would be about 10.7 lbs. per horizontal square foot, instead of 8.25 lbs., which was the weight of the cantilevers.

Whether the supported trusses could have been made as light as the cantilevers, had the shape been immaterial, was not determined; but it is doubtful, as the conditions, particularly the fact that only a small portion of the load was variable, were altogether favorable for cantilever construction.

Expansion.—The author has referred to the fact that only one expansion joint was provided in the trusses, and that in the central span; but in speaking of the intermediate columns as not anchored, he might be understood to mean that it was expected that motion might occur at the feet of these columns from changes of temperature. This was not the case, as it was recognized that the friction from the load on these intermediate columns would be so great as to render motion impossible, even if desirable. The side columns were made very stiff, and were calculated on the assumption that all the wind force would be transferred through them to the ground. Assuming, then, that the trusses were fixed at the outer end, it was expected that the intermediate columns must bend slightly as the length of the end spans varied with the temperature. Allowing for a motion of 1 in. per hundred feet, as the extreme effect of temperature, it was found that the strain caused by this motion in the intermediate column would not add above 25% to the compression caused by the total vertical load, and the combined strain would be only about 12 500 lbs. per square inch.

As a matter of fact, it seems that the allowance of 1 in. per hundred feet was excessive, for it was found that the total change at the expansion joint between a very hot summer day, when, before the covering was all applied, parts of the trusses were exposed to the direct sun, and an unusually cold winter day, before the shed was occupied and partly warmed as it will be by occupation, that is, under a range of 94° Fahr., the maximum contraction at the central expansion joint amounted to only $1\frac{1}{8}$ ins. This is partly explained by the fact that the side posts, instead of being absolutely rigid, as assumed, allow a motion at the top of apparently about 1 in., making a total contraction of about $3\frac{1}{8}$ ins., or $\frac{9}{16}$ in. per hundred feet.

In this connection, the writer can hardly agree with the author's conclusion with regard to the Midway Floor, that "the expansion of each piece of steel is apparently taken up in the riveted joints." It seems more probable that, as the extreme variation in temperature of this portion of the building probably does not exceed 40°, the elasticity of the material is called upon to compensate for any motion which might occur if the material were free to come and go.

General Design.—There are two points in the general design of the

train shed which are not very clearly set forth in the paper or illustrations, and about which a word may not be out of place. Mr. Worcester.

The trusses at the ends of the building, viz., *A*, *B*, *C*, *K*, and *L* are made the full depth of the end, with diagonals each extending over two of the panels indicated by the vertical lines on the cut.

The verticals of these trusses act as beams to carry the wind pressure to the horizontal trusses which are situated in the plane of the top and bottom chords.

The reason for making the main monitor trusses with a 60-ft. span, disregarding the supports which might have been carried to the top of main trusses, was that these trusses are spaced more closely together than the main trusses, those intermediate between the main trusses being carried by the purlins on either side.

Pin and Rivet Connections.—Pin connections were adopted for the main trusses, and for much of the rod bracing, largely as a matter of convenience of erection and to improve the general appearance. There was, however, one very important detail where a riveted joint was used, namely, at the intersection of the bottom chord of the cantilever trusses with the intermediate columns. Here a pin would have been of such large diameter and the necessary restriction in metal of the post so great, to say nothing of the difficulty of having the chords on the two sides of different widths, that a riveted joint was much more satisfactory, and it was adopted without hesitation.

Length of Struts.—One feature of the design which was of considerable importance in the way of economy, but which was somewhat contrary to modern practice, was the ratio of length to diameter allowed in struts in riveted work. If rules such as that "the length shall not exceed 100 times the least radius of gyration" had been adopted, it is safe to say that the purlins and a large amount of the bracing members would have had to be made up of different shapes altogether from those used. The limit adopted was for the length not to exceed 50 times the least diameter. In the writer's opinion, the least diameter in such a limit is a more proper guide than the radius of gyration, as the stiffness of the member, which is the element most to be considered, more closely corresponds with the former. Whether the ratio of 50 is too great is open to debate, but where the members are straightened carefully after riveting, and again looked after when erected, and where, as in a roof, they are not liable to transverse forces of any kind, it seems a wise economy to use a liberal ratio.

Jack Rafters.—It was considered very desirable, in covering the roof, to arrange if possible a truly curved surface which should not show breaks over the purlins, as there are many points of view outside the building from which the line of sight would quickly detect any unevenness. It was therefore with some trepidation that the writer specified for the jack rafters 8-in I-beams, the span being about

Mr. Worcester. 20 ft. This was finally done, but the beams were given a camber greater than that required by the curve of the roof, by an amount (1 in.) which was approximately the theoretical deflection which the beam would have from the dead weight of the covering. The result of this was very satisfactory, the lines of the roof taking a very even curve.

In this connection, it is interesting to note that at the bottom of the slope the pitch is not less than 3 ins. per foot, the steepest which the writer has known of for a similar composition roofing. So far no evil effect has been observed.

Vol. XLIII. JUNE, 1900.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

NOTE.--This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

No. 871.

RIVER HYDRAULICS.

By JAMES A. SEDDON, M. Am. Soc. C. E.

PRESENTED DECEMBER 6TH, 1899.

WITH DISCUSSION.

The following is a brief outline of the methods and results of a systematic study of data taken on the Mississippi River and its tributaries. The system of rivers, the principal drainage divisions and a number of locations to which it will be necessary to refer, are shown on the map (Fig. 1).

As the writer is satisfied that this study has now reached such a point that it will enable river engineers to see clearly what observations should be made and what conclusions might be drawn from them, thereby being the means of saving much time and money, he has ventured to give it as it is, notwithstanding the fact that it contains a number of questions as yet unanswered.

As an introduction, it may be noted that, in working in an established field of science, the relation between phenomena and their fundamental principles is all that need be considered, but it is somewhat different where the phenomena are simply a mass of facts and no fundamental principles are given. In such cases the first step is to find the relations between the given facts; the fundamental principles in which these relations are included will come later.

This is the manner in which this study was made, and its results here presented. The data are subdivided into the different classes in which these simple relations of fact were first sought, and an analysis is then given in which these again are all included in a fundamental relation.

In presenting these relations of fact, each of which has been dug slowly out of great masses of data on various rivers, in some cases covering periods of half a century, the writer has in each case used as an illustration a single instance of the relation in its simplest form, and has noted briefly the variations from this simple form which may occur, and their causes. Thus, the relation between the gauge readings along the river for its own floods is one of these primary relations, while the effects of a flood out of a tributary between any two of the gauges, or the cutting through a bend and the total change of channel near one of them, are considered as variations.

Anyone may test the relation easily, by referring to the published data in the cases given; but, after that, it should be understood that the relation is not to be discredited by selecting other periods, in which it is not so simple, without at the same time considering their variations. The whole conclusion lies often in hundreds and sometimes even in thousands of plates of data, which, of course, cannot be given here, and, so far, the writer must be trusted. The test comes in the application of the final analysis, and, if this bears the test, it will not be necessary for anyone else to go through again the laborious process which he has had to follow.

Slope and Section Data.

It is not the purpose here to attempt to add anything to the data or the theory of velocity-slope relations; nor, indeed, important as they are to the engineer who lays a pipe or builds a conduit, to consider them even directly. Where man makes the form and sets the slope, the resulting flow is the one thing that he wants to know, but where the flow makes its own form, and that a very irregular one, and so distributes its slope that it may be almost all concentrated at certain points at one stage, with little or none there at a different stage, there is not much use in trying to express it as $v = c\sqrt{r\,s}$, for r and s are even more difficult to measure than v.

Of course the fall is determined quite accurately over long reaches,

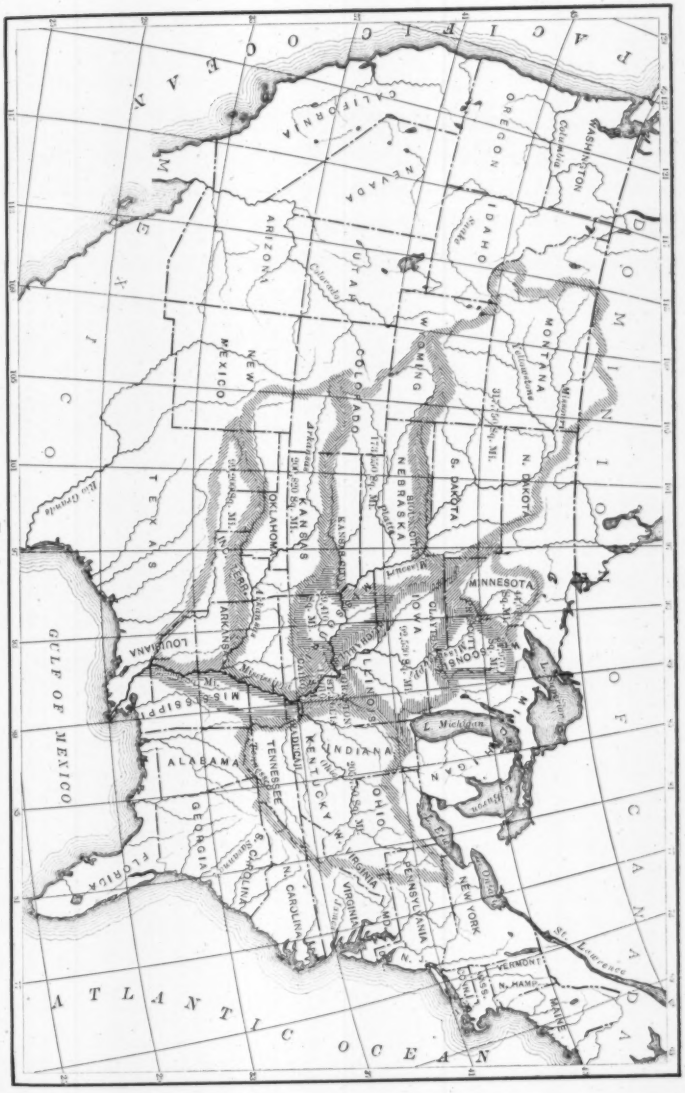

PACIFIC OCEAN
MEXICO
DOMINION OF CANADA
GULF OF MEXICO
ATLANTIC OCEAN
CALIFORNIA
NEVADA
OREGON
WASHINGTON
Columbia
IDAHO
Snake
ARIZONA
UTAH
Colorado
WYOMING
MONTANA
Yellowstone
Missouri
NEW MEXICO
COLORADO
Arkansas
NEBRASKA
BLACK HILLS
S. DAKOTA
N. DAKOTA
TEXAS
Rio Grande
OKLAHOMA
KANSAS
KANSAS CITY
Platte
Missouri
MINNESOTA
IOWA
CLAYTON
WISCONSIN
LOUISIANA
ARKANSAS
Mississippi
ILLINOIS
L. Michigan
MISSISSIPPI
ALABAMA
TENNESSEE
KENTUCKY
INDIANA
OHIO
MICHIGAN
L. Huron
GEORGIA
FLORIDA
S. CAROLINA
N. CAROLINA
VIRGINIA
PENNSYLVANIA
NEW YORK
St. Lawrence
VERMONT
N. HAMP.
MAINE

FIG. 1.

but to determine the values of the mean velocity and the mean hydraulic radius which correspond to it may require a year's observations and survey, while if these are taken in the selected section the slope there may be found to be even reversed, or the water surface may be found to slope in the opposite direction to the flow. It is not at all impossible that, taking a hundred miles of river a mile wide, this formula may in certain cases express the general relations quite as well as it does in 100 ft. of pipe 1 ft. in diameter, but such a relation in the river is wholly useless; while it is equally possible that, if the relation were sought in a 1-ft. length of the pipe, it would be found altogether as erratic as it is for the selected section of the river.

True, there is a difference between the foot of pipe and the mile of river; the theoretical equivalent is probably given more nearly by a relation between resistance and momentum, or the lengths of a fall equal to the given velocities; but, in addition to this, the give and take of stored energy in the river is to be considered. The Lower Mississippi at its bank-full stage has at least a range of from 10 000 to 30 000 foot-tons in the energy of its flow, and in the play of such a balance wheel local slope-velocity relations very naturally are obliterated.

Not only, however, are no such local relations to be looked for in rivers, but also, even if such a general relation should exist at one stage, such a general relation at all stages by no means follows. In many cases the alternations of pool and bar at low water do not differ greatly from a chain of small reservoirs, each emptying through a weir into that below it; and in that case the level and the discharge of each are not affected in the least by the vertical distance which one may be put below the other. True, they may wear under the fall, unless the pool is deep enough and the material is stable enough, but the only slope relation there is this limiting relation with the size of the pools and the character of the bottom.

Where the bed is unyielding and such a condition has simply been formed in the slow process of geological periods, the discharge from pool to pool is no function of slope whatever, but a function of stage. Were the connections plain weirs of given forms, it might be calculated for any level, but as it is more in the nature of a chain of drowned weirs of very irregular forms, the only way to get it is to measure it. In either case, however, it is wholly independent of the different slopes

between the rising and falling river, and it is determined solely by the pool level.

But the concentration of slope between the pools at low water may be entirely reversed at high water, and the narrow sections there control the flow into the series of shallower but broader levels between them; or in this case, the type of control has passed from that of a horizontal weir to that of a vertical orifice, but one is just as arbitrary and just as independent of general slope as the other.

However, in passing from one of these extremes to the other, the river would pass through a stage where something like a general slope relation might be said to control the flow; and here it would probably have a larger discharge on the rising river and a less discharge on the falling river, each in some proportion to the rapidity of the rise or fall. And this is the only point in such a river where velocity-slope relations, even if they could be found, would have any application whatever to its phenomena.

It may be well, finally, to note distinctly that the physics of such a river may be thoroughly known and fully understood, but this would hardly be called hydraulics; for, in the main, it begins and ends in a system of observed facts, and conventional hydraulics throws not a ray of light on the actual proportion in which its elements should stand; they have all simply to be measured.

So much for the river which is not alluvial; what follows will deal strictly with the alluvial river, where the flow handles readily the material of its bed. In such a river, then, that limiting relation referred to, of its slope with its form, is an actual one. In simple cases, this is easily made a subject of experiment. If material from the bed of any such river is put on an inclined plane, a straight channel made through it, and if a given flow which will move the material is turned into it, it will reshape its section until it takes one and only one form for a given inclination.

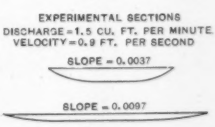


FIG. 2.

Two sections from such a series of experiments made by the writer are given in Fig. 2 to show this effect of slope on a straight channel. They serve very well to illustrate the wide difference in the types of different rivers and its cause, from say, the Platte River at one extreme, where, with a fall of some 6 ft. or more to the mile, it simply runs over the surface of the country, to

the Lower Mississippi at New Orleans at the other, where, with little or no slope, it maintains channel depths 100 to 200 ft. below the Gulf level.

But alluvial rivers are not straight, and neither would these experimental sections remain so. Such an equilibrium is simply a balance of forces, and is not necessarily the only one or the most stable one. Indeed, as the higher slopes are experimented with, the equilibrium of the straight reach is seen very clearly to be less and less stable. It still marks a well-defined point in the experiment at which the flow can be cut off and the section measured, but at the extremes there is really no time when the movement of material in the straight reach can be said to have absolutely stopped everywhere, and it goes on to the formation of bends and crossings, each with more or less accident in its location, with hardly a pause at this, its first and simplest but less stable form of equilibrium.

With a given discharge and slope, the equilibrium of the straight reach has the advantage of being everywhere an identity of form which may be reproduced readily and recognized easily. But though its final equilibrium is not an identity of form anywhere, and its accidental distribution of bends and crossings cannot be reproduced, as a whole, it is no less certainly an identity of effects for the same causes.

But, even with all the freedom in the variations of bend development which the final equilibrium offers, this is but the element of the actions which are going on in the alluvial river. For a given discharge, the bed is everywhere adapting itself to the given slope, while readjusting this slope over pools and bars to its most stable equilibrium, but the river in its season has all discharges from its high-water to its low-water extremes, and different extremes in different seasons; and as these all work each other over, and the results of all are overlaid in the existing conditions at any time, they very naturally produce the endless change of section and the infinite variety of form which are found in the river, the special studies of which are perhaps as fruitless as the study of the differences which may be found in the leaves of the same tree.

To measure this up with anything like thoroughness in a large river, in the first place, requires a great deal of field work, and then begins the difficulty of knowing what to do with it. The general type of the river and the geographical outlines of its bend development

are evident, while the distribution of slope resulting therefrom may be followed roughly through the different forms of the river in its pools and crossings. But, to go further and try to express it in systems of averages, or to follow the general method of compiling it all in the elements of cross-sections, can hardly be said to have been found to be satisfactory, either theoretically or practically. The phenomenon is some form of sequence which no known elements fully express, while to average things which are necessarily different is to obscure their meaning more or less.

However, it is important to note here certain facts in regard to cross-sections. On the Lower Mississippi, where they have been studied most thoroughly, it is plain that the cross-section at any point may not represent the bed of the river there within the limits of some 8 or 10 ft. That is, the mean elevation of the bottom on this line may have such a variation from time to time, while neither the general regimen of the reach nor the surface level of a given stage there have changed perceptibly. Perhaps a special term is needed here to indicate this bed of the river, which, for a given flow, certainly determines the surface level, and it will be called in this sense the "bed in train."

So long as the surface level of a given stage does not change in any reach of river, the bed in train under it certainly remains the same. But when the mean depths of the cross-sections are plotted as elevations of mean bottom on the same profile as this surface, they are generally found to be very irregular; they have sharp ups and downs of some 10 ft. or more, while the corresponding surface changes are measured in tenths or even hundredths of a foot. And, finally, with no change whatever in the regimen of the reach, when one or more of the same sections are re-sounded from time to time, and their mean bottoms show just a like variation of level, there is no room left for the least doubt that the data of sections has simply an accidental divergence from the bed in train of fully this amount.

Indeed, the more thoroughly the bottom of the river is known, the more this accidental variation, which really means nothing, is recognized. At high water sand waves 10 or 15 ft. high may be formed here, while there eddies may be cutting great holes in the bottom; and, with no change in the general conditions of a reach, the bottom at a fixed point in the Lower Mississippi may be found at elevations

differing by as much as 20 to 30 ft., and the mean bottom of a line across it by at least 8 or 10 ft. In smaller rivers these limits are, of course, correspondingly less, but, with their higher slopes, such changes may be even more sudden and erratic.

It is very evident, then, that the bottom anywhere, which at that point represents a real value of the sequence of phenomena in a river

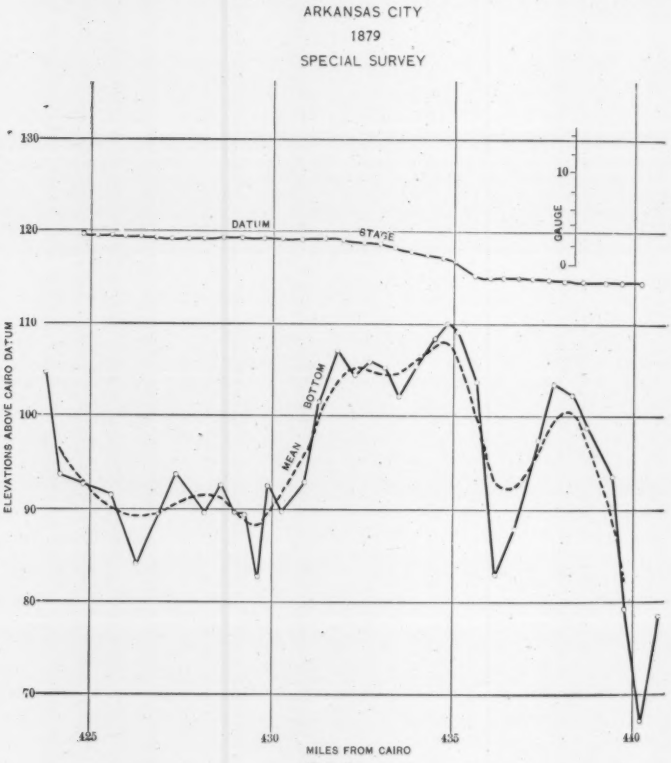


FIG. 3.

is something altogether larger than the bottom on a line there, say half a mile across it, or a like line in any other direction. The local cross-section should doubtless have the most weight, but the bed in train is a part of the condition for some distance, both above and below it.

Some effort has been made to bring the cross-section data together in this form. Taking their elevations of bottom in continuous means of threes, dropping out an upper one and taking in the next lower one at each step, and again repeating the same process on these first means, generally succeeds at last in reducing it to a fairly regular sequence; and it certainly has the merit of bringing the results of a great mass of data together into a very simple form in which they can be seen as a whole and compared with other cases.

The results of a hydrographic survey on the Lower Mississippi in the vicinity of Arkansas City, shown in Fig. 3, will serve to illustrate this. The dashed line of mean bottom is the special sequence deduced by this process of lapped means, while the line joining the plotted points shows the local irregularities in the cross-sections. Each of these, however, in this case, is in general the average of two or more sections often with fifty or more located soundings to each; and, altogether, they have less of this accidental variation of bottom than any of the like data which the writer has yet worked over.

Either of these lines of mean bottom shows the pool and bar formation of this reach very well. It should be noted, however, that probably neither of them shows to its full height the crest of the bar which really controls the level of the upper pool at low water, while in the shorter bars of smaller rivers their failure in this respect would very probably be even greater.

This, however, is about as far as the direct study of form, in an alluvial river, has been carried, and, as before stated, it does not seem that much is to be gotten in this process of trying to understand the river from the bottom up. Practically, every step which the writer has made in river hydraulics, has been made in studies following exactly the opposite process ; but before taking up these, in which the whole point of view will be simply reversed, there is a generalization which the hydrographic surveys give, and which perhaps here it may be best to state.

This generalization rests on the observed fact that the average annual caving in of the banks, over hundreds of miles in some of the rivers, is at a rate which would altogether fill them up in a lifetime; and it follows that, for the river to maintain its channel in such a case, as it does, there must be going on annually some equal and opposite process of clearing this out. This simply considers a lateral action,

whatever the movement of material in the direction of flow may be, when a reach, say like that of the Lower Mississippi from White River to Vicksburg, is taken, nearly a mile wide and some 200 miles long, the difference between the material which comes in at the upper end and that which goes out at the lower would certainly have no appreciable effect on such a prism; while the bank erosion there is not far from a fill of 1½ ft. annually over this whole bottom. The clearing out of this quantity annually is, therefore, necessarily, the work simply of a lateral resultant.

How and when it may act will be better understood later. It is enough here to know that its action is a fact. It is not deduced from experiments on slope equilibriums, nor found directly in the forms of cross-sections. But, certainly in that light, much which has been said about alluvial rivers filling themselves up with the matter which they carry in suspension, is, to say the least, trivial. The difficulty which the water-works engineer has in getting the suspended matter out of the river water when once it has gotten into it, does not encourage the idea that any great additional load will be thrown on the lateral resultant from this cause.

While a river in all its upper part may be cutting down its bed, in the slow process of geological periods, undoubtedly in its lower alluvial part it may be building it up by the same process, and, with its alluvial part relatively short, this action may be even appreciable in the course of centuries; but in systems where the alluvial reaches extend up for thousands of miles, it seems hardly worth considering, and, certainly, not to the exclusion of the equally palpable fact that the alluvial river is also keeping its bed down, or otherwise its channel would fill up by the fall of material into it, and this also in periods which, in comparison with the other action, are relatively immediate.

Discharge and Gauge Data.

Having come now to the point at which is recognized the difficulty of getting anything satisfactory out of the cross-sections to express for a given stage the bed in train, it is proposed, in what follows, simply to take a surface measure for it. Laws of flow are there, whether or not they are known, and in these laws the bed determines the surface. Either can be taken as a measure of the other, and the question of how this measure is to be interpreted can be considered later.

In any case, if the flow into and out of a given reach is the same and the surface level is fixed, there is just so much water there all the time, and the bed, as a whole, is equally fixed under it. In the alluvial river this may be only an instantaneous equilibrium, neither the bed nor the surface may remain fixed, and the surface especially may be changing on account of changes in the amount of water flowing. There are also reasons for thinking that it may change on account of changes in the conditions of flow attending the rise or fall of the river, or the entrance of floods from tributaries; but if none of these will account for a change found in the surface, it must be accounted for by a change in the bed of the river under it.

The first thing required, however, to identify this surface measure, is to determine the flow which corresponds to it. If the river is on a stand through a given reach, with discharge everywhere the same, its surface level may be said to measure everywhere the bed under it; but this alone gives nothing which can be brought into comparison with any other part of the river, or indeed with the same river at another time. But when the discharge is also known, and the reach comes back, say after a flood, to the same discharge with a different surface level, this gives an absolute measure of the changes which have taken place in the bed of the river at that stage, though as yet this measure may not be interpreted.

At any location on the river, where a gauge record is kept, the readings give the surface levels from day to day through all the changes of the seasons, but to use these for the purposes in hand, or as given stages in that river, it is also necessary to know the discharges that go with them. This, of course, is the primary purpose of discharge data and the first thing to be considered in its study.

The observed values of discharge are plotted as abscissas to the gauge readings at the time of the observation as ordinates, and give what is called the discharge-gauge relation there. They show, as a whole, the increasing discharge capacity of the river from low water up, which may be expressed graphically by a mean curve drawn through the plotted points, or mathematically by an empirical equation. This, called in general simply a discharge curve, is taken for the scale of stage at that location. Some of the discharges may be below this mean curve, some above it. Barring errors of observa-

tion, these are different levels of a given stage, and mark either a change there in the conditions of flow or bed changes.

Of course no scale of stage in any river has an absolutely determined zero; the lowest low-water is as far down as it may be followed. In the same way, whether it is given by a graphic curve or an empiric equation, it has a physical limit at the top of the banks, or the level of overflow; it may in cases be more or less continuous above this level, or it may be altogether different. But between these limits it is a given absolute measure of all the average or normal physical and hydraulic properties of the river at that location.

How this scale is to be marked is still a matter of convenience. At any point in the river it is simply a range of level between low and

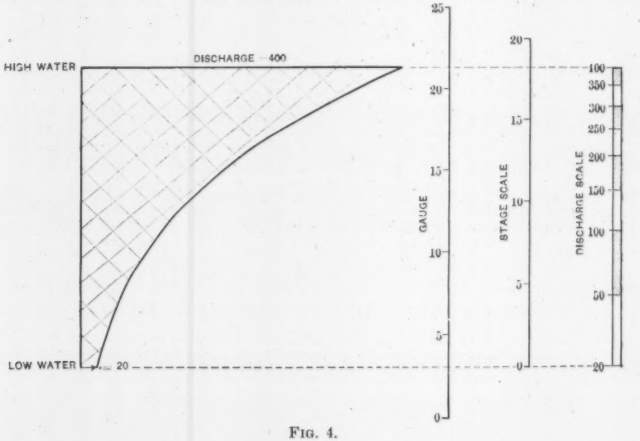


FIG. 4.

high water, with a given range of discharge corresponding to it. Drawn with a general characteristic of the Missouri River, but without any necessary reference to any river whatever, it is shown in Fig. 4, with some of the forms of marking which have been taken in different uses of it.

Where either the gauge or stage scale is used, the normal discharge curve corresponding to it is given at the beginning, and after that the stage is simply taken in feet and fractions with the given values of discharge understood. The discharge scale has the advantage of bringing together the two elements which define the given stage there, but it does not give their coincident values so accurately

everywhere. On it here the discharge is marked in thousands of cubic feet per second, a unit in which it will be generally expressed.

While each of these methods of marking the given stage has, as will be seen, its special field of uses, in the immediate study of the data, the gauge, of course, is the first one that is taken; but from the first, also, this must be clearly distinguished from simple gauge readings, for they have in them as well all the variations which may occur in the surface level of a given stage there through the flood season. It is true that, as this scale of stage differs materially from point to point in the same river, it may also change more or less from time to time at the same point, but what it is must be gotten before such changes can be followed, and it is important, in determining what it is, not to mix it up at first with all the sequence of bed changes and different conditions of flow which follow each other in infinite variety and endless succession from season to season. These have been called simply changes of plane, without regard to their causes, and in what follows they will be so designated.

It may be recognized here that the term "stage" is used, in a highly technical sense, as a standard of measure in rivers. Without a measure for force or for heat but little progress could be made in those sciences, and some definite measure is equally necessary in the hydraulics of alluvial rivers. In its general use, stage refers to that range in a river between high and low water, as high-stage, mid-stage and low-stage, or, in terms of gauge readings, 10, 20 and 30-ft. stages. But these are plainly very different things in a river say 100 ft. wide and another 1 000 ft. wide. This ordinary use expresses but one element of something which has at least another dimension. In terms of slope, area and mean hydraulic radius it has three, with none of them known and no known relations between them; but these are all summed in the coincident value of discharge, which, observed anywhere for one stage, is everywhere on the river for the same stage, the same value.

Expressed in this scale of stage, the different parts of the river may be brought into immediate comparison, and also the different types of different rivers; but as yet it has had no special reference to any particular river, and has hardly touched upon the data from which it is to be determined. The mean curve drawn through the discharge-gauge relation has been noted as giving it, but this alone forms only a

part of the data by which it is determined, and indeed a very small part in its extension over a whole river and the study of its general relations there.

Of course, this scale of stage gets its absolute value finally from discharge observations, but were not the relations in which all the different scales of stage stand to each other in different parts of the river first known, the study of discharge data might very imperfectly determine it; and these relations are almost wholly a question of the gauge data.

Between tributaries, whenever the river is on a stand, it is plain that its surface simply marks its points of equal stage on all the gauges ; or, referring to Fig. 4, the low-water reading on the gauge is everywhere the zero of the stage scale with a corresponding discharge of 20. The top of the flood, in the same way, may mark the equal high-water stages, and intermediate stages are continually being given, between the varying rises and falls, in this direct manner.

In all this it will be found that the different scales of stage in the same river in general stand to each other in a simple ratio. That is, these readings, on any two gauges taken as ordinates and abscissas, give a relation between them which is a straight line. And further, in any rise or fall of the river, when a suitable time is allowed to let the difference of stage pass from the location of the upper gauge to that of the lower, the readings so taken mark the same line through the intermediate and recurrent stages and in all the periods of rising and falling river.

Fig. 5 will serve as a simple illustration of this. On it the Missouri series of discharge observations, taken in 1879 at St. Charles, are shown plotted to their St. Charles gauge readings, with the mean discharge curve given by them there for that season. With this also is given the relation between the gauge readings at St. Charles and those taken at Hermann, 69 miles up the river. In this the time interval of $\frac{3}{4}$ of a day is allowed for equal stages to reach St. Charles from Hermann, and interpolated St. Charles readings, this much later each day, are taken for ordinates to which the Hermann readings are plotted as abscissas.

All the readings on these two gauges, from early in April to the fall low-water in November, are shown in the plotted points of this gauge relation. And it is plain that for all values of discharge corresponding to the St. Charles stages, accurate corresponding Hermann values

may be taken immediately from this line. Or, the gauge relation is said to transfer the St. Charles discharge curve to Hermann, and this it does also with altogether about the same precision with which the curve is originally determined for St. Charles. Thus, in 1879 there are 170 discharge observations at St. Charles to give the mean curve there, while, at the same time, there are somewhat more than 200 entirely independent observations of the surface level at St. Charles and Hermann to fix this relation between them. Either determination has its possible

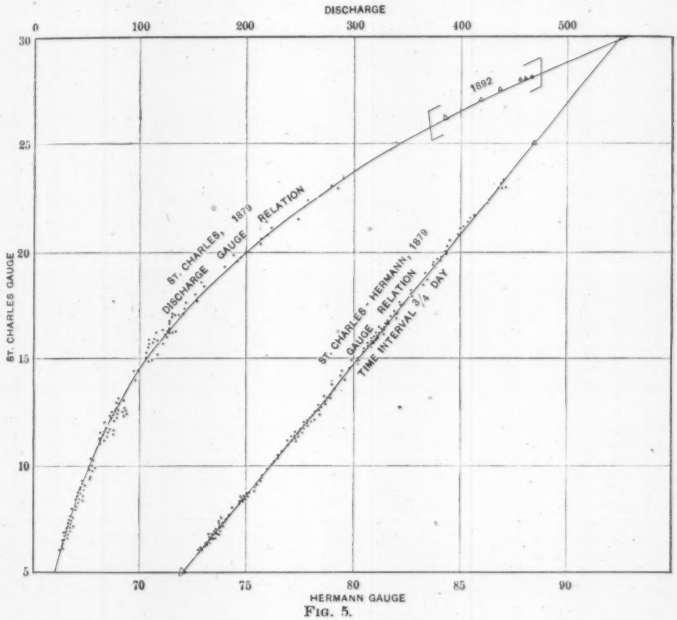


FIG. 5.

error, but the resulting scale of stage at Hermann may not have more but actually less of this error in it than the St. Charles determination.

As the St. Charles discharge curve is transferred to Hermann, so again it may be transferred from gauge to gauge along the river, meeting a positive test of both the original curve and of all the transfers at every other location where discharge observations have been also taken on the river in the same period. The only other observations made in this year on the Missouri were a short series at Sioux City, some 780

miles further up, and a transfer and check over this distance is a long, and necessarily a very careful, process. In passing such tributaries as the Platte or Kaw Rivers special studies of the gauge relations there for years are also necessary, and need either some discharge observations on the tributary, or discharges of the main river above and below it.

In fact, it is only in the same river that the relations between the scales of stage are straight lines, and "the same river" is, of course, to be understood as a physical term, and not a geographical name. The Missouri above and below the Platte need not have been the same river, but essentially it is, and this relation practically holds on the Missouri, at least from Sioux City to its mouth. Indeed, the smaller tributaries further down, in their low-water periods, have not more than 0.1 or 0.2 ft. of effect on the gauges below them, even at the extreme low-water stages of the main river, and this, of course, amounts to little or nothing at the higher stages.

However, a flood out of these tributaries is another matter. The Osage in extreme flood may add something like 150 to the discharge of the main river, and it is plain that in such periods, the line of relation, between the scales of stage above and below it, is no longer given by the gauge readings. The flood does not necessarily affect the scales of stage at either of the locations, unless perhaps the upper one is within the range of back-water action from the tributary, but the marked excess then of the lower gauge readings must show in a corresponding divergence from the line of the ordinary gauge relation, and, in fact, on the Missouri River, this is used as a measure of the floods from the tributaries.

Such cases are, of course, to be met with in plotting the gauge relations, and are excluded in these transfers of the discharge curves. Some of them come at inconvenient times, but long low-water periods are the ordinary conditions of these tributaries, and, using only such periods in general, the transfer may be carried across them without sensible error. In the same connection, also, the investigator will soon find that he has to deal with more or less gauge data which should never have been published. The effect of the tributary flood is continuous, and this fact shows in the gauge relations which follow it; but false readings are not, and by that they are marked at once on the gauges where they are made. The line selected for Fig. 5 was free

from all such complications. It is not at all an exceptional gauge relation, but it must not be assumed that all gauge relations are by any means so simple.

So far, the scales of stage have simply been taken as marked on the gauges, and, whether an original discharge curve, or a discharge curve transferred, they serve only as a measure of a more or less limited reach of the river at the given location. Before considering a more general condition, it is convenient to pass to the second form of marking shown in Fig. 4, and called the stage scale. For this, 5 ft. on the St. Charles gauge is arbitrarily taken as a low-water standard, and 25 ft. as a high-water standard, corresponding in this year to the discharge values of 19.6 and 347, respectively. In Fig. 5 the corresponding levels on the Hermann gauge are shown as determined on the line of that gauge relation. And taking these corresponding points from St. Charles to Kansas City on all the gauge relations between them, in all the combinations in which they may be plotted, gives for this year the equivalent gauge readings called low and high water, respectively, in Table No. 1, at the locations of each of the given gauges.

TABLE No. 1.

Location of gauges.	Mid-Bank Distances.	Year, 1879.		Equal stages (h) (high water).
	Miles above St. Charles.	Equivalent gauge readings.		
		Low water.	High water.	
St. Charles	0.0	5.0	25.0	20.0
Cottleville Landing	16.5	25.2	40.1	14.9
Washington	40.5	46.4	62.3	15.9
Hermann	69.0	72.0	88.5	16.5
Fisher's Landing	87.5	88.9	106.0	17.1
Jefferson City	112.2	110.7	127.5	16.8
Providence	136.5	134.2	150.0	15.8
Boonville	160.4	156.0	172.0	16.0
Glasgow	187.0	178.5	196.5	18.0
Miami	222.5	212.7	230.7	18.0
Waverly	249.0	233.7	251.0	17.3
Lexington	270.0	253.7	272.8	19.1
Camden	283.5	267.0	283.0	16.0
Missouri City	307.0	285.0	302.7	17.7
Kansas City	332.7	307.0	325.4	18.4

The differences between these arbitrary high and low waters, called equal stages, mark, of course, the scales of stage at each of these locations, mid-stage being simply the half of these values, and so on, for all intermediate stages. But the fact that these are 20.0 ft.

at St. Charles, and only 14.9 ft. at Cottleville Landing, 16.5 miles up the river, or 19.1 ft. at Lexington, with 16 ft. at Camden, 13.5 miles above it, shows distinctly the marked longitudinal variation which may be found in the river; and gives, at the same time, its definite measure. The whole length of the river is certainly a sequence of such variations, in its extremes at least as large as the differences found in these cases.

This sequence, in its actual longitudinal form at any time, has not yet been determined. But, while in certain bends it may stand unchanged for a long time, in general, it is not fixed. What was 20 ft. at St. Charles, in 1879, was but 18 ft. some five years later; in which case the river there had left its old channel around the St. Charles Island, and shortened its course by several miles. In the same way, the 16 ft. at Camden, in time, may take the place of the 19.1 ft. at Lexington. And this sequence, in its time variation, may be followed with precision in the discharge and gauge data. It is one of the hasty conclusions of a superficial study of this matter, that there are no gauge relations because they are not fixed ones, while, on the contrary, it would certainly be very surprising to find such a longitudinal variation in the river actually tied to all its gauges.

However, in the Missouri in general, and it is inferred in other northern rivers, there is really a period in which there are no such relations. Of course, when the river is frozen over there is not much rise or fall to mark any relation, but it seems that an ice-bound surface may result in a different distribution of slope from that of a free surface, and, in the place of the river standing at the same level on all the gauges, it may, in such a case, change in a perfectly arbitrary way at any of them. The clearing out of the ice also probably cuts up its bed pretty badly, as the forming and breaking of ice gorges might be expected to do. But certainly during and immediately following this period no relations between gauge readings may be found. That it soon readjusts itself and all its flow, in all the variations along its course, and, in all its changes from high to low water, is again found held in this chain of simple stage ratios, is not at all inconsistent with what has preceded it, and on the whole marks no less but rather more the presence of a determined and controlled equilibrium.

And, finally, the larger changes noted in the equal stages, which are in general a matter of years, must also be supplemented by

smaller changes which may be only a matter of seasons. Not only may the river change its whole course and its whole form at the location, but the bar may be built further down and bring the gauge from the level of a lower into that of an upper pool. The first may have an effect of several feet, the second may be limited to fractions of a foot, but on the Missouri the second may also mark several distinct gauge relations in the season, or throw a part of the plotted period out of the line of the general relation. These, of course, mean different scales of stage there for each period, but in such details the scales of stage are certainly restricted to very limited reaches. This might be of value in special studies, but, as a general river measure, the average is all that is wanted; and where there are two or more such changes in the season, a mean is taken for the gauge relation.

This outline will show the general processes and results of a study of discharge and gauge data. It gives an absolute measure of the river at a number of different locations, which probably cover all its variations, and follows with precision all its changes at each of them. To close for the Missouri, it is only necessary to add the fact that the Missouri in different years is a different river. Or, to state this technically, the level of the zero of its general stage scale is found to change materially between a sequence of high-flood and low-flood seasons.

Taking, for instance, the fall low-water of 1883, following a succession of phenomenally high-flood years, the low-water level corresponding to the discharge of 19.6, in the place of being the gauge readings from 5.0 at St. Charles to 307.0 at Kansas City, given in Table No. 1, would be found to average about 1 ft. lower. Whether or not the high-water stage is affected in this is still unsettled; from such data as there are on the subject, and from the nature of the change, it is inferred that it is not; in which case all the values in the column of equal stages would then average 1 ft. more. In the same way, the fall low-water of 1895, following a succession of phenomenally low-flood years, is about 1 ft. higher, giving values in the column of equal stages averaging about 1 ft. less.

The process of such changes will show later. They arise from the counter-actions, already noted, of erosion which would fill up the bed, and of that lateral resultant which would keep it down. Every cutting bank and bar is caving into the river continually, in

this way throwing upon the bed material which is not carried away, for there is no evidence of an increasing movement of material down stream; and indeed, accumulating as it would, it could not be thus taken up in the flow without soon overloading it; therefore, it must simply be spread over the bottom in some way, and this, at all times,

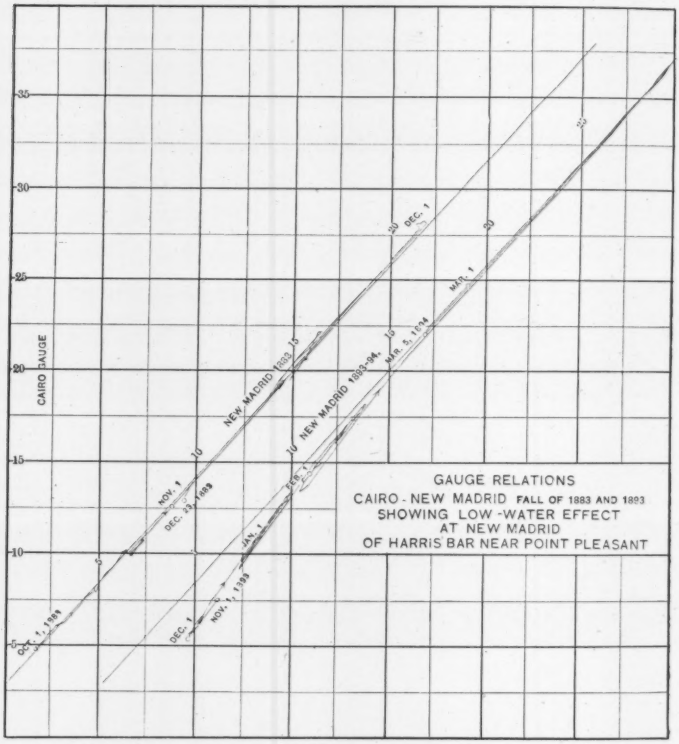


FIG. 6.

below the existing water line. It is really, then, only the flood which can move this material up on the slopes of points and bars; and the extreme flood is required to throw any of it altogether out at the bank level. Thus there is no doubt that the action of the series of extreme floods is this general clearing out of the low-water bed, and of the

series of low floods a corresponding filling up, notwithstanding the fact that this has not yet been observed in anything but these stage measures.

With all that has been given in this study of the Missouri, the subject may be followed now into the data of the Lower Mississippi; and it will be enough to note briefly the special features which the given processes show for this river, and at the same time any new problems which its marked physical difference and greatly enlarged scale may

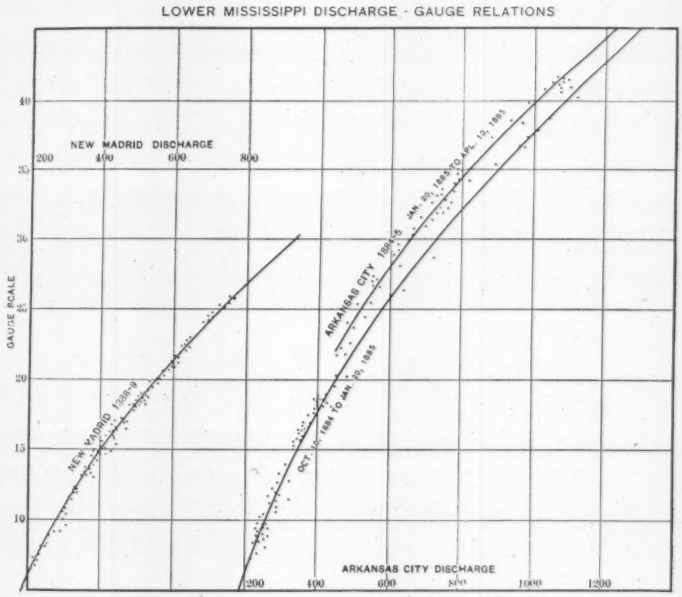


FIG. 7

bring into view. And, certainly, between a river of the size of the Missouri, and one whose floods have some three times the extreme discharge and twice the rise, this matter should be fairly covered for alluvial rivers.

Figs. 6 and 7 serve as illustrations. The New Madrid discharge-gauge relation in Fig. 7 does not differ essentially from that of St. Charles on the Missouri, and it may be seen by the Cairo-New Madrid gauge relations in Fig. 6, that, in general, it may be transferred in just

the same way in which St. Charles was transferred to Hermann. But it will be noted here also that, while the relation at the higher stages has not changed perceptibly in 10 years, a decided but limited range of bar effect has come into the surface levels of the New Madrid low-waters. And it is this pronounced stability of these relations, some of which now go back nearly 50 years, subject now and then to these low-water variations, disappearing between the 10-ft. and the 20-ft. stages, which may in general be said to be the special features of the Lower Mississippi gauge relations.

This low-water form, however, is not a variable but a fixed condition in the 300 miles between Red River and the Gulf. Even as high up as Red River, the Gulf level is some 10 to 20 ft. above the mean bottom of the Mississippi, and through all this reach it acts upon the low-water surface in much the same way as did the crest of the bar in 1893 at New Madrid. For any such reach, of course, the variation of the scale of stage through it is a special and a single problem in each case.

But the marked exception of the Lower Mississippi gauge relations to the general features noted above, is found since the times, and at the locations, in which its great floods have been held within levees. It began to be seen, more than 10 years ago, along the swamp fronts, where the large natural outflow was being materially checked by the building of levees, that a general lowering of the low-water level was an accompanying phenomenon; and it was soon inferred that the levees there, producing abnormally large floods, like the series from 1881 to 1883 on the Missouri, were simply clearing out the low-water bed in the same process. Indeed, it was here that the action of that lateral resultant of the movement of material in a river was first suggested, even before the magnitude of erosion, determined between hydrographic surveys, had made it so plainly a necessity, and before the reverse process had been tested in the filling up of the Missouri River bed after the succession of low floods in 1895. Simply in these levee effects was it seen that, whatever might be the longitudinal movements in the building up of the bars at high water, it was, after all, the magnitude of the flood volume which kept down the level of the low-water plane; and the somewhat homely illustration suggested itself that, as a whole, the flood went through the river somewhat like a plow.

All these features of the Mississippi, however, raise no essentially new questions; but, the Arkansas City discharges, in Fig. 7, do. The

marked difference in the level of the discharges between the first and second periods is a change of plane of a character which must be considered here. If it was simply a permanent local change at that point, it would naturally be assigned to some radical change in the regimen of the reach, similar to that noted at St. Charles, when the range of stage there changed from 20 ft. to 18 ft. But, far from being such a permanent change, it is not even a local one.

As may be seen in the example given, the gauge relations on the Mississippi are not only especially stable, but they are also, in general, especially well-defined single lines; and, in that case, every variation which shows in the surface levels of the same discharge at one of the gauges, must show in the given ratio and the given period at the other. Changes of plane are transferred as well as discharge curves, and, as far as the gauge relations below Arkansas City, each shows a single line for both of these periods, this change of plane is necessarily an identity all the way down.

It is, of course, only actually identified at the location of the lower gauges; but where it is found repeating itself in this way, in both its ratio and period from gauge to gauge down, there can be little doubt that it is simply continuous between them. And in this process the change of plane shown at Arkansas City, is found to be a continuous one as far down as it can be tested accurately, and at least, in this case, for over 300 miles.

This change of plane coincides roughly with a flood out of the Arkansas River, it does not show on gauges above, and it is practically continuous through the rest of the flood and as far down as it can be tested through the rest of the river. While such a change of plane certainly holds a difference in the general slope from above, this difference does not account for it, for this difference is the thing itself. In the same way the difference of slope between the rising and falling river does not help matters, for this difference is as great at Helena as it is at Arkansas City. And, in fact, this change of plane is found here, as repeatedly elsewhere, to have much the same magnitude between the two periods whether the river is rising, falling or stationary in either of them. Indeed, as was pointed out at first, not much dependence is to be put in the direct slope-effect in the flow of rivers; it appears and disappears continually, as a general slope controls or does not control the flow.

Such a continuous change of plane, in the case in hand, has been considered, probably in part at least, a change in the conditions of flow accompanying the flood out of the Arkansas. And, for the entrance of floods from tributaries, there is this to be said, they certainly raise the surface level of the river above them without any corresponding increase of discharge. In cases in the Lower Mississippi this may be followed up with precision on the gauge relations; thus, floods out of the Arkansas River have been seen distinctly to raise the surface level of the Mississippi over 40 miles above by more than 1 ft. The flow of the main river, then approaching the tributary with its momentum somewhat reduced at a continually increasing rate, might, perhaps, be expected to carry a higher plane below for some time and some distance. And, indeed, in such a case as the run-out of the Lower Mississippi from Red River to the Gulf, where, even in the highest flood, only a small fraction of the water in the river is above the Gulf level, and the great mass of it has not a foot-pound of potential energy, such effects of floods from Red River certainly seem to have startling proportions.

In the years of great floods, also, changes of plane of this character, are marked, from Cairo down, and, as changes in the conditions of flow, would be assigned to the large return flows and back-water drainage following the extended overflow. And, as they do not show here to any great extent in the low-flood years like that of the New Madrid discharges given, some such assumption is not altogether unwarranted. But, at the same time, it should be recognized clearly that marked bed changes also occur between these periods. The raise to the top of the flood has the effect of the erosion through the greater part of a season at its surface, with all the bar-building forces of the river under it, while this is practically reversed in the fall. And, until this movement of bed may be determined, it is needless to discuss further the special causes of changes of plane of this character.

Of course, with such changes of plane in the Mississippi, there is little doubt that changes of plane of the same character occur in the Missouri, but, if they are not large enough to be seen there, they need not be considered. The same change in the conditions of flow which would change the surface level by 1 ft. in a river from 50 to 100 ft. deep, would only change it by 0.1 ft. in a river from 5 to 10 ft. deep; and in the different type, the reduced scale, and much more active bed

changes of the Missouri, changes of plane of this character are simply unimportant phenomena. In the same way, again, if such studies were carried into a river like the Platte, all the importance of the Missouri River gauge relation might disappear in the small range and continuous bed changes of that river. Hydraulics is limited to relations which show; and perhaps it is altogether possible to study with a microscope, the flow in the simplest form of flume, in such detail as never to understand it.

Finally, all these detailed studies of the discharge and gauge data in a river may be put together in the discharge scales, and show at a glance all the floods of a river for years, with the combinations which make up such a flood as that of the Lower Mississippi.

Figs. 8 and 9 illustrate the use of the discharge scales. The regular gauge readings are plotted from day to day as ordinary hydrographs, and the discharge scales are drawn through the flood season from low-water up. Arranging these, then, in order down the river, all the contributions from the tributaries show in the differences between the flood volumes from gauge to gauge, and may be checked readily at any point with summations of the tributary increments taken from the gauge relations, while the combined floods of two rivers, such as the Missouri and Upper Mississippi, are seen summed up in the flood volumes of the Middle Mississippi at St. Louis.

In these diagrams simply the average discharge scales have been used, and no account has been taken of changes of plane through the period, but, of course, in checking up the final discharge curves in the different rivers it is necessary to take account of these changes. And there, indeed, the whole study of these data in the rivers meets one of its severest tests. If the sum of the St. Charles discharge and the Grafton discharge do not make the St. Louis discharge there is something wrong. Individual discharge observations may have their errors, but correct scale of stage can only have changes of plane and wrong gauge readings.

All this, however, may be followed, well enough for an illustration, in the average scales in Fig. 8. Thus, the top of the 1881 flood is a little less than 500 at St. Charles with a little less than 350 at Grafton, while St. Louis shows about 800. But the plane at Grafton is raised by an up-stream effect from the extreme flood in the Missouri, Grafton discharge observations at that time showing this distinctly, and, allow-

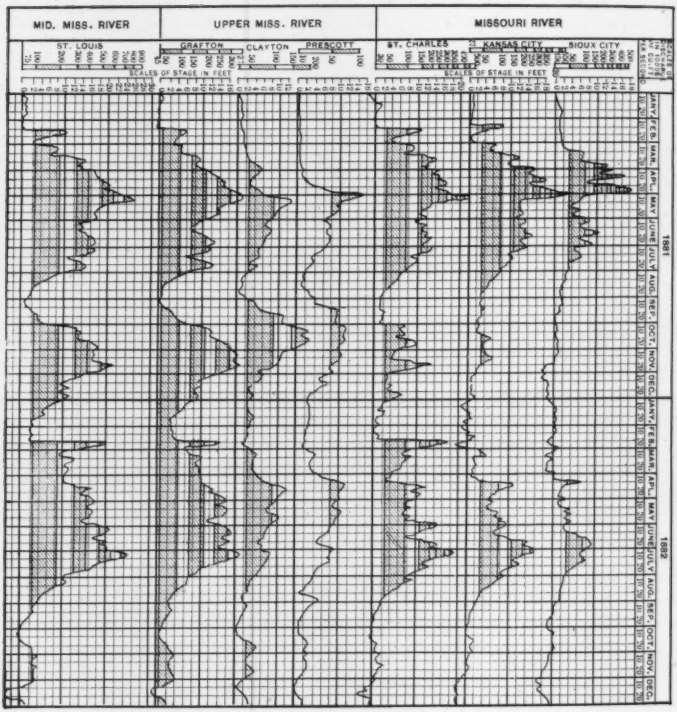

MID. MISS. RIVER
UPPER MISS. RIVER
MISSOURI RIVER
ST. LOUIS
GRAFTON
CLAYTON
PRESCOTT
ST. CHARLES
KANSAS CITY
SIOUX CITY
SCALES OF STAGE IN FEET
1881
1882
JAN'Y.
FEB.
MAR.
APL.
MAY
JUNE
JULY
AUG.
SEP.
OCT.
NOV.
DEC.

Fig. 8.

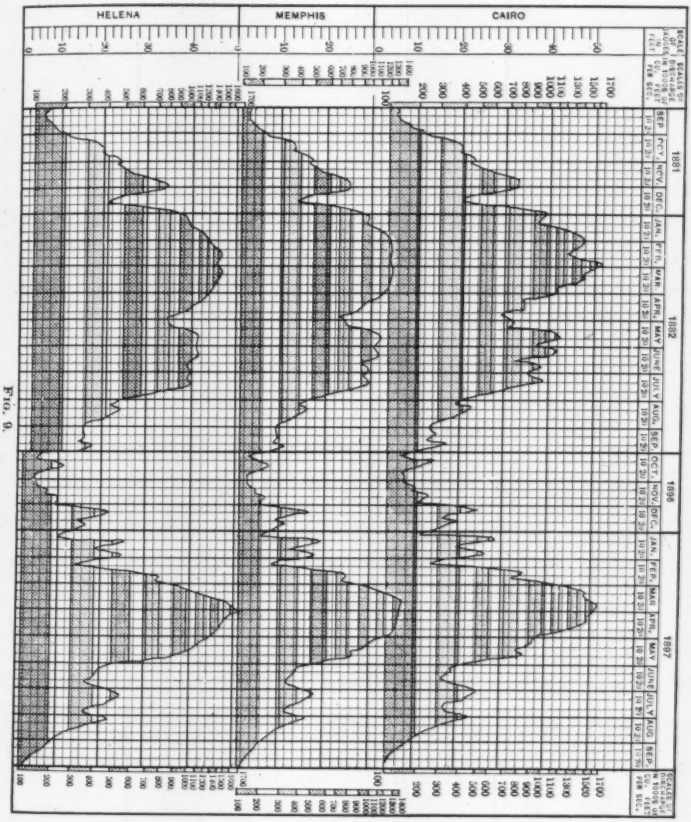


FIG. 9.

ing for this, it checks very well with St. Louis and the discharge from the Missouri. The next low water is 50 at Grafton, and a little below 50 at St. Charles, differing about the same amount from the 100 at St. Louis. And again, the top of the 1882 flood is 350 + at St. Charles, and 350 + at Grafton, with 700 + at St. Louis. At the same time, the irregularity of the ice period in these rivers is marked, and particularly in the forming and breaking of ice gorges from Sioux City down, in the spring of 1881.

Fig. 9 gives, in the same way, from Cairo to Helena, two of the greatest floods in the lower Mississippi; and Fig. 10 is an outline map showing the river's system of swamp basins. In general, all along these basins the ground slopes back from the river into an area of low swamps threaded with lakes and bayous, and draining into such rivers as the St. Francis and Yazoo.

As the Mississippi rises above its bank-full stage, the flood water is drawn off into these upper swamps, to be returned to the river at their lower ends after an interval. Fig. 9 shows this action through the first or St. Francis basin. The total volume of the flood, not only given by the Mississippi stages, but also traced down from its sources through the combinations of the upper rivers, is shown in the discharge scale at Cairo, while the same flood is shown at Memphis at about its point of maximum outflow, and again at Helena, where it receives the return flow from the St. Francis basin. However, the total return flow is not shown at Helena, since the outflow into the upper Yazoo at the same time is necessarily lost to its stage and discharge.

The former condition of free overflow is shown in the year 1882, and the first effect of the levee system now in progress in that basin is shown in 1897. And though the system was far from complete at the time, and was badly broken in the flood period, still the increase in stage and discharge is most marked. While the flood extreme of 1 700 at Cairo in 1882 is reduced by outflow to but little more than 1 200 at Memphis, and comes back with inflow to about 1 400 at Helena, in 1897 it reached about 1 400 at Memphis, before the breaks in the levee relieved it, and came back again at Helena to the Cairo value of 1 700.

This will illustrate the method of handling the extreme-flood data, which, as before noted, lie beyond the range of regular discharge curves. While not pretending to represent the actual discharge at any spe-

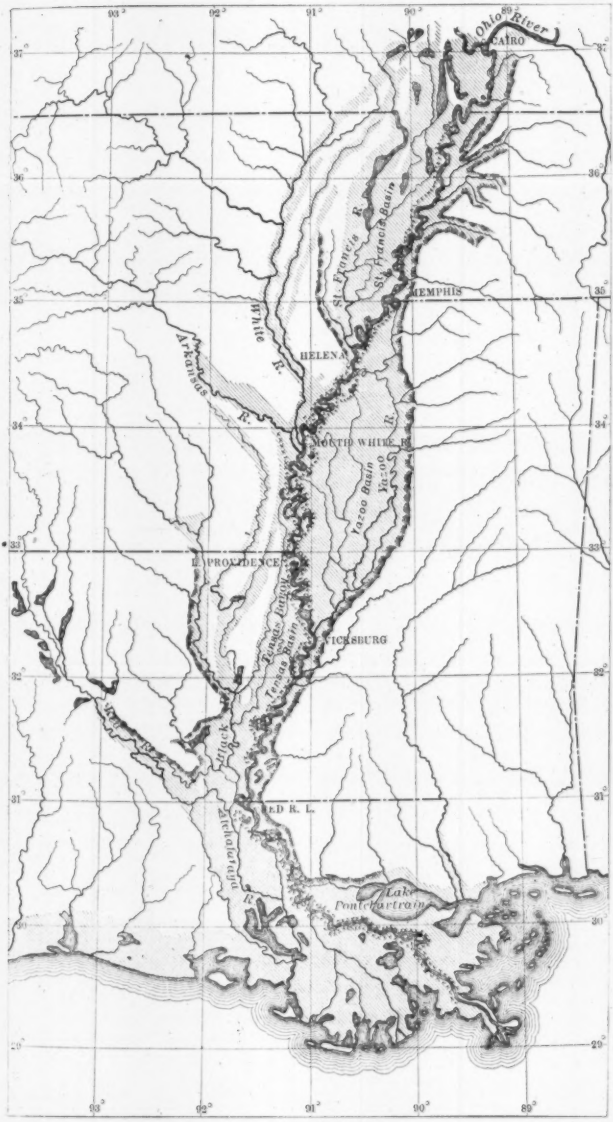

Ohio River
CAIRO
MEMPHIS
HELENA
WHITE R.
ARKANSAS R.
St. Francis R.
St. Francis Basin
NORTH WHITE R.
9300 Basin
Yazoo
VICKSBURG
PROVIDENCE
Tensas Basin
Atchafalaya
RED R. L.
Lake Pontchartrain

FIG. 10.

cial point in the flood, these scales, as an average, bring all the data together and compare their total volumes in a form in which some reasonable consistency may be recognized. However, further down the Mississippi, and especially since the construction of levees, the discharges are often so irregular, between the draw of a large crevasse on the one hand and the lake-like character of points in the general overflow on the other, that they can hardly be followed through the flood period, even in this system of rough averages.

Finally, Cairo and Memphis, in Fig. 9, show very well the fixed character of the general Mississippi regimen. The 15 years from 1882 to 1897, has brought no marked change there; while at Helena, in the same period, a case of the levee effect before noted is given. Here, in 1897, a glance shows that the discharge, as brought down to Cairo, from the rivers above, and as found repeated at Memphis, cannot be continuously some 50 less at Helena, as the 1882 discharge scale would give it. The precise lowering there is a matter of the gauge relation; but nowhere is the general fact more plainly seen than on the discharge scales.

In following the discharges down from Cairo to Helena through this period within banks, their readings on the discharge scales are essentially the same as they should be; for, in general, the tributary increments here would not show on these scales. Thus, the crest at the end of November, 1881, is a little less than 800 from Cairo to Helena, followed by a fall to just 400, and a rise to about 1 000, where the limits of overflow begin to show. But it must be noted also, that this does not mean that the actual discharges for the same scale readings in different periods are necessarily the same. The scale readings are normal values, and, so long as the actual discharges differ from the normal by the same amount all the way down, their normal readings will be the same.

This general identity from Cairo to Helena, therefore, is not only an identity of the discharge, but also an identity of plane. That continuous plane, noted in the case of the Arkansas City discharges, shows all through the data of the Lower Mississippi. It is referred to here again in order to note finally that whatever its causes may be, it has, from point to point down, exactly the same flood sequence preceding it, and its accurate recurrence from gauge to gauge only marks it as a very general and a very definite phenomenon; it does not add essentially to the difficulty in its explanation.

Flood Movement.

So far, it has simply been noted, in connection with the gauge relations, that, when the river was rising or falling, a suitable time was to be allowed for the given stage to pass from the location of the upper gauge to that of the lower. The precise determination of this time, from the gauge readings themselves, forms the subject of this study of flood movement.

In the rough, this time interval is seen very well in the hydrographs of any river simply by noting the difference in the times at which the flood crests show in order down on their respective gauges, and it is not only marked at the crest, but, somewhat less distinctly, at the foot of each rise. Thus, in Fig. 8, in the period May-June, 1881, on the Missouri, the general interval of about seven days from Sioux City to St. Charles is fairly indicated in the time between these points marked by the various crests and hollows; though it may be more or less blurred in the case of any one of them by the changes of plane and tributary increments which show in this period. In the same way the special retardation at the top of the April flood of that year will be noted. And, passing to the Upper Mississippi, its much slower rate of flood movement is shown most distinctly. Thus, comparing the movement from Prescott to Clayton with that from Sioux City to St. Charles, in the Mississippi, the flood takes about the same time to pass over about one-fourth the distance.

In the Lower Mississippi, Fig. 9 shows an interval of some three or four days from Cairo to Helena, and if any one should undertake to follow up this interval on the discharge scales, through all the rises and falls of the river, by noting the difference in time between the same discharges at Cairo and Helena, he would be struck at once with the general accuracy and the constancy of this interval, not only for the crests and hollows, but also for all the floods within banks.

But days are far from being the measure of precision which is wanted, and to obtain this, very different methods are needed. It is evident, on the face of it, that the time required for a given stage to pass from an upper to a lower gauge is determined with the greater precision the greater the difference of stage which is passing. Thus, the periods of rapid rise or fall are those which must be used for precision, while the period is hardly defined at all in the slower changes

of stage at the flood crests; though, taking these crests as a whole, symmetrically, it may still be seen in them. Almost any interval at the crest will give the gauge relations, while the line between any two gauges is given without any interval at all through all the stationary periods of the river which extend to both of them; but where a rise or fall of some 2 ft. a day occurs, an error of a tenth of a day in an assumed interval begins to make distinct divergences from the line of the gauge relation.

Having the line of the gauge relation given, and plotting such rises and falls with assumed trial intervals, the divergence of each plotted point from this line, of course, marks the error in the time assumption at that stage; but as, in the great mass of the data, the gauge relation and the time interval have to be determined together, it is convenient to have in mind, clearly, from the first, certain standard types which different trial intervals must give to the form of such a plotted relation.

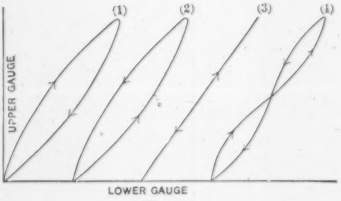


FIG. 11.

Types of trial time intervals are shown in Fig. 11. The first is an interval altogether too short. If, in plotting a trial relation, the assumed interval was less than the true time, then, on the rise, the lower gauge values taken would not have had time to rise to their equal stages, and would, therefore, be too small in proportion to the rapidity of the rise. In the same way, on the falling stage, the lower gauge values would not have had time to fall enough, and would, therefore, be too large in proportion to the rapidity of the fall. The trial relation would, therefore, plot as in (1) where the rising and falling stages are indicated by the directions of the arrows. In the same way, if the trial interval for the flood is taken too long, the lower gauge values would be too large for the rising and too small for the falling stages, giving the trial relation shown in (2). Between (1) the interval altogether too short, and (2) the interval altogether too long, the relation of the flood can always be reduced to one or the other of the forms (3) and (4). The form (3) shows very plainly that this interval for the whole flood is a constant, or that the flood movement does not vary with the stage; while all conditions of a variable

flood movement are included in the form (4). And here, in the special case indicated by the arrows, it represents a flood movement which increases about uniformly with the stage; that is, the mean interval is about as much too short for the lower part of the flood as it is too long for the upper part.

By more or less trial plotting of selected floods, the character of their movement, between any two gauges, may be recognized, and, in case the interval is a constant, it may generally be determined in this way to the nearest tenth of a day. It is also plain, however, that any flood out of a tributary entering between the gauges in this period would destroy altogether the simple form of the relation shown in (3). Indeed, unless such periods were recognized and excluded from the study, they would lead to very serious errors in both the rate and the character of the flood movement.

It is also necessary to allow for those changes of plane which may occur from time to time at the location of one or the other of the gauges. To determine the time required by a given stage to pass through the reach it must be taken on the level which it has in passing. So long as the levels of the given stage remain the same at the two ends, or change together, as they do through those continuous changes of plane noted on the Lower Mississippi, the relation will give the form (3) when the correct time interval is taken, but when the level of the given stage changes at one of the gauges without a corresponding change at the other, the form of the true gauge relation should shift in the same way; and an interval which might make it approach more closely to the form (3) would not then be a more correct interval, but an erroneous one.

These, of course, are the same variations which have been met in the transfer of discharge curves, and which, with a little care, do not affect sensibly the accuracy of that process. But here, in the determination of an element which is marked alone in the form of special periods of rise and fall, they are much more serious difficulties. Thus, taking gauges above and below a tributary, after getting by trial a correct interval between them, and then, with this interval, systematically plotting their relation through years, a number of the floods might be seen to correspond very closely with the form (3), and a number of others might be marked clearly as affected by floods from the tributary, or changes of plane, and rejected. Between these two,

however, there would still be a number of periods in which it would be uncertain whether their form did not measure as truly a flood movement there as did the form (3), and in which, if they did, it would be plain that the flood movement, in that case, was not at the same rate as the other, nor indeed any longer a constant in its character.

This, of course, would have been all well enough if it had proved that the flood movement varied from time to time and for different forms of floods, but when the precision of the determination is also considered it is at once seen that in itself it simply does not prove anything. There is no use in taking interpolated values between the daily gauge readings any closer than to the nearest tenth of a day; if for no other reason, because it is doubtful whether the observers have actually read their gauges closer to the set time, if indeed they can be counted on for that. With an interval then of some two or three tenths of a day between the given gauges, this relation showing in any case the form (3) means only that the flood movement at that time is this given rate and is constant within the limits of some $\pm$ 20%, while in the other case it means that its rate is something more or less and variable within the same limits. Thus, the two cases simply are not precise enough for their contradictions to mean anything.

Again, while the short reaches lack precision in this time element, the long reaches lack precision in the equal stages. When the river is rising or falling 2 ft. a day, a change of 0.2 ft. in the stage is equal to a tenth of a day in the time interval, while in periods of less rapid rise and fall the time equivalent is correspondingly greater. In a long reach, therefore, covering a number of tributaries, the summations of their increments, small in themselves, may affect materially the apparent time of the passage of a given stage through the reach; and even where they may not do this, their combinations tend to curtail greatly the periods which are practically free from this difficulty, until finally, it is not once in years that a suitable period for the determination of its flood movement can be found; that is, a period without change of plane between the gauges, all the tributaries being at a fixed low-water, with a sharp rise and fall of the main river.

To meet all these difficulties in the precise determination of the flood movement, the system of plotting the data, called extension-gauge relations, was planned. Its principle consists of studying a

flood movement in the form of a number of gauge relations, between the same upper gauge in each case, and lower gauges following in order down; showing, in close succession, the movement of the same flood from the same start, through a series of successively lengthening reaches. The form of each gauge relation, then, in its order, includes all the flood movement of the relations preceding it, with such additional movement as the flood may have in that case between its lower gauge and the gauge next above it.

An instance from the Lower Mississippi extension-gauge relations is given in Fig. 12, covering a reach of 277 miles of that river, from Cairo down. In all this the time intervals have been determined first by trial. Cairo gauge readings are taken uniformly as the ordinates,

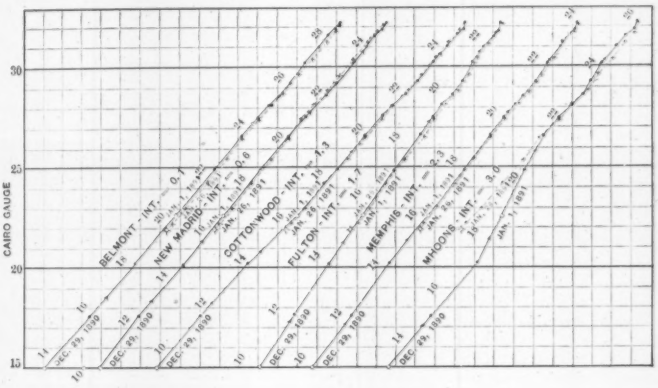


FIG. 12.

and interpolated readings, later by the intervals, respectively, as abscissas, and the gauge relations, Cairo-Belmont, Cairo-New Madrid and so on, to Cairo-Mhoons, are plotted in succession. Altogether, they leave no doubt whatever as to the movement of this flood. The Cairo form is simply reproduced in a given ratio on each of the gauges after the lapse of a fixed time.

Gauge readings, of course, contain a tenth or so of accidental error, and no single one of these relations reproduces exactly the Cairo form in the given ratio at the location of its lower gauge; but when the divergence is in one direction at one gauge, and in an opposite direction at the next, there is certainly no change of form shown in

the passage of the flood through the reach; and, indeed, in this case it is repeated even more exactly at Memphis, 230 miles below Cairo, than at Cottonwood 123 miles below, or Belmont only 21 miles below.

Thus, it is not necessary, in every case, to see this flood movement extending down to Memphis, or to Mhoons, in order to recognize it. If it is seen extending down to Cottonwood, and if, from there on, more or less increment from tributaries comes in to break it up, it is still seen, and the reason that it does not show further down is recognized. Thus, a part of the rise at Fulton may be marked with an excess of Fulton stages, and in that case the same part at Memphis and Mhoons would at least be marked with the same excess, though more probably it would actually show increments somewhat greater. Through all this, however, there would be little doubt that the same flood movement was still there, if outside conditions had only left the same flood in the river to show it.

The Mhoons' relation is given here as an instance which needs a special notice. On January 1st (January 4th at Mhoons), the gauge reader reported that his gauge was washed out, and that readings were taken on temporary gauges. This continued until May 23d when his gauge was reset, the inspection showing that his readings were 2.2 ft. too low. It is very evident from the relation that this error was made at the first setting of his temporary gauge, and if it is so taken the flood movement is shown to be very fairly continuous to Mhoons. But if, on the other hand, the error had been distributed over the whole period up to the inspection, as is often done before the gauge readings are published, it would have given an altogether incomprehensible surface movement of the river there for about five months. This is simply a caution which should be borne in mind continually in any use of the gauge data.

It is this process of recognizing, in the extension-gauge relations, what does not, as well as what does, show the flood movement, which gives all the certainty to its determination from the ordinary gauge data. The method may be said to be almost mechanical. The intervals of the various reaches are determined by trial. Stages between the readings of the lower gauges are then interpolated by these intervals, and are each, for the given day, the effect of the given reading on the gauge at the head of the reach, in so far as this has been

determined approximately by trial. These are then plotted in succession, showing the whole surface movement over hundreds of miles, everywhere as nearly in the one relation of cause and effect as a single time adjustment will give it ; and all, for the better part of a year, are brought finally under the eye at once, and in quick succession a number of years can be examined. After that the whole matter is simply one of intelligent seeing.

Such a flood movement as that of Fig. 12 is seen frequently and through long periods, extending over the shorter reaches, but with little precision in the time intervals; while, again, through the long reaches, with the much smaller percentage of error in the intervals, the flood movement can be followed only through shorter or less frequent periods. But as far as it is so recognized and followed, so far it is determined. Its actual rate is not given precisely between any of the gauges, but it is there, or it would not be found precisely in their summations of it.

In the case given, the movement may be said to be recognized down to Mhoons, and the interval there determined to the nearest tenth of a day, or to within $\pm$ 0.05 in a total interval of 3 days. This is less than 2%; and again, where the same movement is recognized down to Helena, or through a total interval of 3½ days, the limit of error is then reduced to less than 1.5%; and noting repeatedly just such a flood movement in different floods and through all the different stages within banks, it comes to be held as a very general phenomenon of the river, observed with a good deal of precision, and certainly determined more closely than might have been expected simply with interpolations between daily gauge readings.

This, however, is all a case of the constant interval, or the flood movement which does not vary with stage. Type (4), Fig. 11, or the flood movement which varies with stage, is recognized and its form is followed in the extension-gauge relations in much the same way. That is, as the further a flood goes which does not change its form, the more certainly is it seen that its form is unchanging; so, in the case of a flood which changes its form, the further it goes the more distinctly this shows itself in its greater divergences from the standard form at the head of the reach. Thus if Type (4), Fig. 11, showed a perfectly uniform increase of flood movement with stage for 100 miles, in 200 miles it should everywhere show just twice the divergence.

However, the case of a variable flood movement is never quite so simple, for it represents a condition which cannot continue indefinitely. Thus, the upper part of a flood cannot go on moving much faster than the lower part without coming to a point where it would topple over; and, before it reached this, any uniform or normal movement which it might have at a given stage would certainly be modified by the abnormal form which the flood was taking.

It is this fact which makes the determination of the flood movement in the Missouri, from the ordinary gauge data, so intricate. In the Missouri it has probably as definite a normal character as that of the Lower Mississippi, but this, in the Missouri, is a very decided increase with stage, and the normal movement, therefore, is modified at least by limiting forms of the floods, and is subject to variations with their changes. This, with the fact that interpolations between daily gauge readings are necessarily a less precise surface measure in its short and sharp floods, and that its tributaries through long reaches have a much larger proportionate effect on the main river, all taken together, deferred anything like the setting of values for the normal flood movement in the Missouri to a much more advanced stage of the study.

It is, however, in this contrast that it should, perhaps, be noted that a given flood movement in a river is not a law but a fact. Even with the general character of the constant interval on the Lower Mississippi, there are still a few specially sharp floods and some short periods in the transitions from stationary to flood conditions which seem to be exceptions. But that does not detract at all from the significance of the fact that its whole surface movements from low water up for almost its whole time are certainly held in this given fixed relation.

As simply an empiric constant, running through the infinitely complex and ever-changing conditions of flow, this recurrent flood movement seemed to be the one thing in that river which did not change; but why it should be of the same rate at high water as at low, while the mean velocity of the river for the same change about doubled its value, or why it should be over 400 miles a day from Baton Rouge to Carrollton when even the high-water velocities there were less than 100, were questions at the time well calculated to puzzle anyone; for probably everyone would first try to explain this flood movement by these observed velocities.

Such considerations may have some value in finally knocking out of the investigator himself any remnant of a delusion that water moves in threads and filaments; but as these considerations lead to nothing in the matter in hand they are here omitted. This flood movement is simply taken as it is, a well-determined fact of the river, which, as a fact, naturally rests on some property of the river as a whole which is altogether as general and as strikingly fixed as the flood movement. Anyone is, of course, at liberty to try and find this property in observed velocities if the total independence between the two, in the cases just cited, does not satisfy him.

Yet, with all the river absolutely determined in its scales of stage, and such a property of the river as disclosed by this flood movement also given, it seems certain that somewhere there is to be found some equally absolute relation between them. But seeing this and getting the relation are quite different matters, and it was not until the writer had studied analytically the case of flood movements in a chain of lakes or reservoirs* that he began to be able to calculate some of the elements of a river from its given discharge curves and flood movements. The methods at first were crude and laborious in the extreme, and need not be considered here further than to note the final step in this chain of investigation, for from here on the question will be taken up as an immediate deduction of a fundamental physical equation of rivers, with simply examples of its application.

Mathematical Analysis and Computed Regimens.

It is plain, of course, that for a given difference in discharge between the upper and lower ends of any reach, the rate of rise or fall in the reach is determined solely by the area of its water surface at that time. This general relation is given by the equation.

$$(Q_1 - Q_2)\, d\,T = A\,(d\,h) \qquad \text{I}$$

where Q_1 = the upper and Q_2 = the lower discharge in cubic feet per second, T = the time in seconds, A = the surface area in square feet, and $(d\,h)$ = the mean rise in the reach on a scale of feet in the differential interval, $d\,T$.

So far, this simply expresses the fact that a cubic foot of water requires a cubic foot of space, and expresses the condition of the whole of a river as it does every other form of variable flow. For the

* *Transactions*, Am. Soc. C. E., Vol. xl, p. 355.

river, the special problem is to express the terms in this condition in elements which may be determined independently.

Now, in whatever condition of rise or fall any river may be, $\left(\frac{d\,Q}{d\,h}\right)$ is the rate of change of discharge with its change of stage, and $\left(\frac{d\,h}{d\,T}\right)$ is the rate of change of stage at any point of its flood oscillations. The change in the value of discharge in an assumed interval $\Delta\,T$ is then $\left(\frac{d\,Q}{d\,h}\right)\left(\frac{d\,h}{d\,T}\right)\Delta\,T$. Now, whatever this may be on either rise or fall, there is some point down stream at a distance $\Delta\,l$ where the value of discharge at the end of the interval $\Delta\,T$ is just what it was at the upper point at the beginning of the interval. The difference in discharge between the upper and lower ends of the reach, of length $\Delta\,l$, is then $\left(\frac{d\,Q}{d\,h}\right)\left(\frac{d\,h}{d\,T}\right)\Delta\,T$; and the surface area of this reach is $W\,\Delta\,l$, where W is its mean width in feet. Substituting these, respectively, for $Q_1 - Q_2$ and A in equation I gives

$$\left[\left(\frac{d\,Q}{d\,h}\right)\left(\frac{d\,h}{d\,T}\right)\Delta\,T\right]d\,T = W\,\Delta\,l\,(d\,h),$$

or

$$\left(\frac{d\,Q}{d\,h}\right)\left(\frac{d\,h}{d\,T}\right) = W\,\frac{\Delta\,l}{\Delta\,T}\,\frac{(d\,h)}{d\,T}.$$

In this form $\left(\frac{d\,h}{d\,T}\right)$ and $\frac{(d\,h)}{d\,T}$ are not necessarily identical, for $d\,h$ in the first is the rise in the interval $d\,T$ at the head of the reach and $(d\,h)$ is the mean rise over the whole reach during the same interval. As, however, the corresponding magnitudes of $\Delta\,l$ and $\Delta\,T$ are reduced, the difference between $\left(\frac{d\,h}{d\,T}\right)$ and $\frac{(d\,h)}{d\,T}$ may be made smaller than any assignable quantity and finally disappear from the equation as a differential when $\frac{\Delta\,l}{\Delta\,T}$ becomes $\left(\frac{d\,l}{d\,T}\right)$.

Hence,

$$\left(\frac{d\,Q}{d\,h}\right) = W\left(\frac{d\,l}{d\,T}\right).$$

Now, by its definition here, $\left(\frac{d\,l}{d\,T}\right)$ is the flood movement at that point of the river, or the rate at which equal stages or discharges move down; and, representing its value in feet per second by m, the equation becomes

$$\frac{d\,Q}{d\,h} = m\,W. \qquad \text{II}$$

It may be noted, in the above process, that the parentheses ( ) have been used to indicate definite physical values determined independently. Thus $\left(\frac{dQ}{dh}\right)$ is given by the discharge curves. On the Lower Mississippi it may vary from 10 000 at low water to 60 000 at high water; or in the Missouri from 3 000 to 40 000; and it is commonly referred to anywhere as the change of discharge per foot of rise or fall there. Again, $\left(\frac{dh}{dT}\right)$ is the rate of rise or fall at any time. It is in common use as feet per day or inches per hour, but, being expressed here in second units, its value, for a rise of 2 ft. per day, is $\frac{2}{86400}$. And, finally, the $\left(\frac{dl}{dT}\right)$ is that rate of flood movement which was at

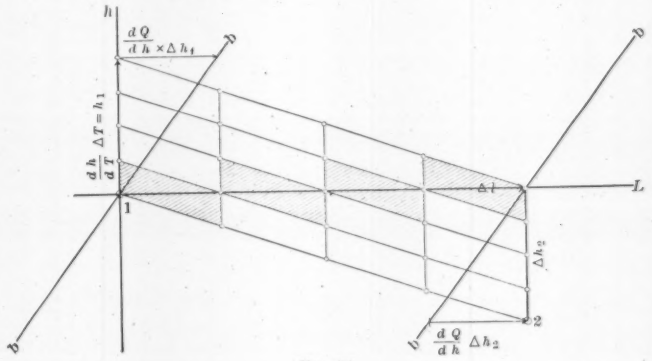


FIG. 13.

last determined with precision, at least for long reaches, in the preceding study of that subject.

With these in mind, the process of deduction is simple enough, and it is seen readily that Equation II is a most general equation of rivers, true for every element of any river from high to low water and from one end of it to the other. But, before going further, it may perhaps be as well to give, in addition, a somewhat more distinctly physical deduction of it.

Taking then, as in Fig. 13, an origin O_1 on the surface at the upper point at a given stage, and the axis of L a length of the river below at the same stage; its level representing the surface slope of the river when the discharges are equal at the two ends, or the river is on a stand

all the way down; then, with the ordinate h measuring the change of stage from this stand, the actual surface of the reach in any given rise is shown by the line $O_1 O_2$. Taking now $b\,b$, an element of the discharge curve for the reach, its absolute value everywhere at the level $O_1 L$ being Q_1, and the rate of its change of value $\frac{d Q}{d h}$, then, if ΔT is the interval which it takes to rise from O_2 to the level $O_1 L$ and Δh_2 the corresponding change of stage there, at the beginning of this interval the upper discharge is Q_1 and the lower is $Q_1 - \frac{d Q}{d h} \Delta h_2$, the difference being $Q_1 - \left(Q_1 - \frac{d Q}{d h} \Delta h_2 \right) = \frac{d Q}{d h} \Delta h_2$. In the same way, at the end of the interval ΔT, the upper discharge is $Q_1 + \frac{d Q}{d h} \Delta h_1$ and the lower discharge is Q_1, the difference being

$$\left(Q_1 + \frac{d Q}{d h} \Delta h_1 \right) - Q_1 = \frac{d Q}{d h} \Delta h_1.$$

The mean difference in the discharge between the upper and lower ends of the reach, through the interval ΔT, is therefore $\frac{d Q}{d h} \left(\frac{\Delta h_1 + \Delta h_2}{2} \right)$, and there is thus brought into the reach during this interval a quantity $\frac{d Q}{d h} \left(\frac{\Delta h_1 + \Delta h_2}{2} \right) \Delta T$ more water than is carried out. This must fill the volume indicated by the vertical area $\Delta l \left(\frac{\Delta h_1 + \Delta h_2}{2} \right)$ multiplied by the width of the river W.

And equating these two

$$\frac{d Q}{d h} \left(\frac{\Delta h_1 + \Delta h_2}{2} \right) \Delta T = \Delta l \left(\frac{\Delta h_1 + \Delta h_2}{2} \right) W;$$

or, as before,

$$\frac{d Q}{d h} = W \frac{\Delta l}{\Delta T} = m\, W. \quad \ldots\ldots\ldots\ldots \quad \text{II}$$

The application of this equation may perhaps be best noted by bringing it into comparison with the ordinary hydraulic formula

$$v = c \sqrt{r s} \quad \ldots\ldots\ldots\ldots \quad (1)$$

$$\frac{d Q}{d h} = m\, W \quad \ldots\ldots\ldots\ldots \quad (2)$$

Thus (1) and (2) have in their order essentially similar physical measures. For instance, v in the river is obtained by measuring the

discharges from high to low water and dividing them by the corresponding areas of their cross-sections; while $\frac{d\,Q}{d\,h}$ is obtained by plotting them to the corresponding gauge readings and differentiating the resulting curve. In the same way, s and m are taken from surface observations; one being a rate of changing level, the other a rate of advancing stage; while r and W are simply characteristic lengths, the first a sort of inverted radius, or area divided by wetted perimeter, the second a plain surface width.

Finally, also, in applying the formula $v = c\sqrt{r\,s}$ to a perfectly general form of flow, where everything may have any arbitrary variation in a given length, subject alone to this condition, even if it expressed everywhere such a variation and the c in it was an absolute constant, it would still be very evident that the element through which it must be taken is not a uniform length element. Thus, if it is to be summed into an equality and averaged, the given length is to be divided into a certain number of elements, each $v\,\Delta\,T$ long, and in each of which it expresses an equality, and the general relation of the whole distance is the average of these elements. That Equation II has exactly the same characteristic may be seen readily by taking it before its reduction, as $\frac{d\,Q}{d\,h}\,\Delta\,T = W\,\Delta\,l$. For $W\,\Delta\,l$ is simply the surface area in the variable elements of length, and $\frac{d\,Q}{d\,h}\,\Delta\,T$ corresponds to the $v\,\Delta\,T$ for each of the constant intervals of time into which the distance may be arbitrarily divided.

Here, however, the parallel ceases, the ordinary hydraulic formula refers in the main to conditions in which the discharge is fixed, such as the discharge of a pipe under a given head, or a conduit with a given form and slope; and it is certainly a most necessary relation there, notwithstanding the fact that its c is a very intricate variable, for Equation II has no application whatever to such a case. But, on the other hand, in rivers Nature offers a condition of an almost continually varying discharge, and, as there is no way by which an irregular volume can be measured more accurately than by taking the amount of water required to fill it, in such a field the fundamental relation between dimensions and the action on them of these varying in-flows and out-flows is not only applicable, but is the only one, in the range which may be covered by it, to be considered.

But, of course, this is not the place to discuss the application of either of these formulas to all the conditions of flow to which they may be applied. And, in fact, in the case of Equation II, there are probably a number of applications which the writer has never yet thought of. He has been interested mainly in applying it to alluvial rivers, and even there, in those reaches in the Lower Mississippi, where the gauge relations are no longer straight lines, it has been already intimated that its application in each case was a special and a single problem. In the whole of this lower river also, with its peculiarly constant flood movement from high to low water, marked with decided differences from reach to reach, there are some questions on which the writer is still working. The only case, therefore, which will be offered here is its application to the general regimen of the Missouri from Kansas City to St. Charles.

The 1879 scales of stage have been given for fifteen locations on this reach, in the former study of discharge and gauge data, and whether they should cover equal time intervals, or simple intervals of distance, is not here a practical question, since their longitudinal sequence has never been determined on this river, and a plain average of them is all that can be obtained, and also, very probably, all that is here necessary.

The average of the equal stages in the tabulation referred to, is 17.1, and the equation of the St. Charles discharge curve in terms of its stage scale is $Q = 0.15\ (h_s + 19)^4$ where Q is the discharge in cubic feet per second and h_s is the St. Charles stage taken from a zero at the low-water level of 5 ft. on that gauge. Calling h_m the average stage of the reach from Kansas City to St. Charles, it is plain that

$$h_s : h_m = 20.0 : 17.1 \text{ or } h_s = \frac{20.0}{17.1} h_m$$

and substituting

$$Q = 0.15 \left(\frac{20.0}{17.1}\ h_m + 19\right)^4$$

from which

$$\frac{d\,Q}{d\,h} = 1.117\ (h_m + 16.3)^3.$$

The corresponding value of flood movement on the Missouri has not yet been given; and indeed there were a number of questions in flood movement which were purposely left to be answered after Equation II had been determined.

Solved for this element, it is $m = \frac{1}{W} \times \frac{d\,Q}{d\,h}$, and with this it only takes a glance at the discharge scales in Fig. 8 to see why the flood movement is so much slower in the Upper Mississippi than in the Missouri. In the same way the marked retardation at the top of the great flood of 1881 on the Missouri is simply the transition from channel filling to valley filling. That the value of W changes sharply in such a transition is plain, while the especially irregular forms of the over-flow discharge curves have been noted. And, in fact, it seems probable that if these forms are ever to be determined satisfactorily it will not be done by measuring their discharges, but by measuring the widths of over-flow and the flood movements, and calculating them.

Again, the 400-miles-a-day flood movement from Baton Rouge to Carrollton is simple enough to anyone who knows the physics of that part of the river. It is well down in the run-out, where the range of high water is being reduced from 50 ft. at Red River to nothing at the Gulf; and the value of $\frac{d\,Q}{d\,h}$ is correspondingly increasing; while at the same time it is also an exceptionally narrow river. It is clear, then, from the equation, that values of m will be found here out of all proportion with its rates elsewhere.

And finally, the constant rate of flood movement from low to high water in the Mississippi will be recognized here as simply a special relation between its general discharge curves and its form. So long as $\frac{d\,Q}{d\,h}$ and W vary in the same proportion m must be a constant, and that they have such a variation vertically in the Mississippi is a fact of its form, and that they do not have such a variation there longitudinally is another; while in the Missouri they have no such special variation together in either direction. But still, all the Missouri scales of stage are linked together in a chain of gauge relations, and there is little doubt that this river has fully as definite a form of its own to correspond with the $\frac{d\,Q}{d\,h}$ given by them, and that its own especial flood movement is there, as certainly as it is in the Mississippi, even though it cannot be determined so accurately in its gauge data.

There is, however, a special variation of m which must be considered. It has been seen, of course, in the deduction of Equation II, that the elements of rise or fall, of whatever magnitude, cancelled out in the final equation, leaving a relation which was immediately

independent of them. This does not mean that the value of m is absolutely independent of the rate of rise or fall, but simply that it is changed by it only as the $\frac{dQ}{dh}$ may be changed by it, assuming, at the same time, that the element of form W at the given level remains the same.

This is a variation of the flood movement from its normal, of the same character as a change of plane in the discharge data, and probably this is the only surface measure in which all those variations of discharge at the given level can ever be followed. It must be looked to also to account for those exceptions to the general flood movement noted in the Lower Mississippi, and the much more frequent changes of the same character in the Missouri. But it should be distinctly recognized simply as a variation from normal flood movement, and it does not coincide with the normal discharge or the normal $\frac{dQ}{dh}$ determined for the river.

It was in this light that the specific determination of the normal flood movement in the Missouri was taken up. The extension-gauge relations from Kansas City to St. Charles had all been plotted for a period of 15 years, showing the character of the whole surface movement there; but it did not seem worth while then to go further in order to pick out the values of its flood movements in all their variations. But when it was understood very clearly that there was a normal movement there which rested on its general form, already so distinctly marked in its discharge scales, the case was different. The extension-gauge relations were then examined and the movements at those periods least affected by tributaries, and best suited to give normal values for different stages, were taken and averaged for the general relation.

This result, expressed in miles per day and taken on the same stage scale in which the $\frac{dQ}{dh}$ of this reach has been given, is

$$m = 70 + 3.27\, h_m.$$

It is thought to be correct to at least within 3 or 4%, but it does not show such limits as distinctly as in the case of the Lower Mississippi. It is the best that could be done with data very imperfectly suited to its determination. Its simple and precise determination in such a river will require other data than those of the ordinary gauge readings.

Equation II, when m is taken in miles per day instead of feet per second, becomes

$$\frac{d\,Q}{d\,h} = \frac{5\,280}{86\,400}\, m\ W,$$

$$\text{or, } W = \frac{16.364}{m}\,\frac{d\,Q}{d\,h},$$

and substituting the values of $\frac{d\,Q}{d\,h}$ and m gives finally, for the equation of the general regimen of the Missouri from Kansas City to St. Charles,

$$W = 16.364\,\frac{1.117\,(h_m + 16.3)^3}{70 + 3.27\,h_m}.$$

This, of course, is simply an equation of dimensions. It gives the value of W from high to low water, and, as such, determines the dimen-

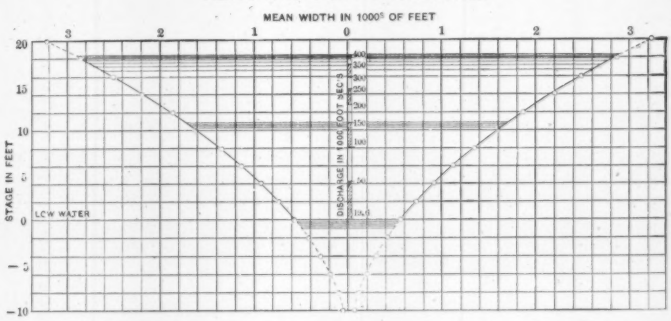


FIG. 14.

sions of the mean hydraulic prism through this reach of river. But no co-ordinates of position have been taken anywhere, and therefore any showing of its mean cross-section must have a further assumption; for any number of areas may be taken between two levels, all having the same value and the same variation, but all differing in their actual forms.

Assuming that W increases symmetrically from an imaginary center line, Fig. 14 shows the mean cross-section of the prism for this reach. Of course, in a divided channel, the actual increase of W with stage, might be altogether toward the center, while at low water especially, its increase would be in any and every direction; but it does not make any difference in what direction it increases, so that the amount and rate of its change are seen distinctly, and this is shown very well in Fig. 14.

Above the water line of the 400 discharge, the river is more or less out of its banks, and that is the limit of this section. What $\frac{dQ}{dh}$ and m may be above that is not known, but it is known that they are not continuous. In the same way there is no guarantee that they would be continuous if they could be observed below low water; but this section is not either of them, but a form of the river, which would be there just the same if all the water on the instant were taken out of it, and this relation has been carried down to 10 ft. below low water, as the most probable continuation of this form as it approached its lower limits.

Between these water lines, however, this regimen of the Missouri is simply measured; not, of course, directly, but as definitely as a distance may be measured with a base and two angles. This measure also has in it probably about the precision of ordinary cross-section data; but it brings out a marked form of bed which all the direct cross-section data has never even suggested. Indeed, it is only in the late surveys on the Lower Mississippi, where the section lines were leveled over all bars and up to the high-water limits, and large means made up from these, that types of this character began to be shown. That the surveys would not give the type accurately is self-evident; for any artificial method of making up the means from them can hardly fail to average values which belong to decidedly different stages. But in the summations of these surveys the essentially triangular character of the Lower Mississippi regimen is well brought out, which its constant flood movements and second-degree discharge curves also give for it.

It is this type of regimen which represents plainly the general equilibrium between the erosive and bar-building forces of the given river; and the type in the Missouri is certainly sufficiently striking. Unlike the plain triangle of the Mississippi, its curvature shows that its forces tend to cut a sharper central thread and build up a widening bar area as the stages are higher. The fact, also noted before, that the low-water level in the Missouri was especially susceptible to the varying forces found in the sequence of high and low-flood years, is here made very plain. Considering the erosion, its action practically consists in cutting down the high banks into this central thread or deeper portion; for the bottom, at the foot of a cutting bank, is rarely above low water. All the caving, therefore, from the cutting banks and bars falls into this central thread and through the low-water season it all stays there,

while the flood period can never fill any of it back above the stage to which the flood reaches.

It is clear that, in the succession of low-flood years, this normal regimen may be changed very materially, this central thread filled up considerably, with all the widths of the lower portion reduced, and the widths of the upper stages increased correspondingly. While again, to maintain the general average regimen of the normal, the action of the succession of high-flood years will be a variation of regimen, from the same causes, but equal and opposite in its character.

Fig. 15, in which the different regimens of 1883 and 1895, before referred to, are given, will serve to show this action. Neither of these regimens has the data needed to determine it absolutely, and they

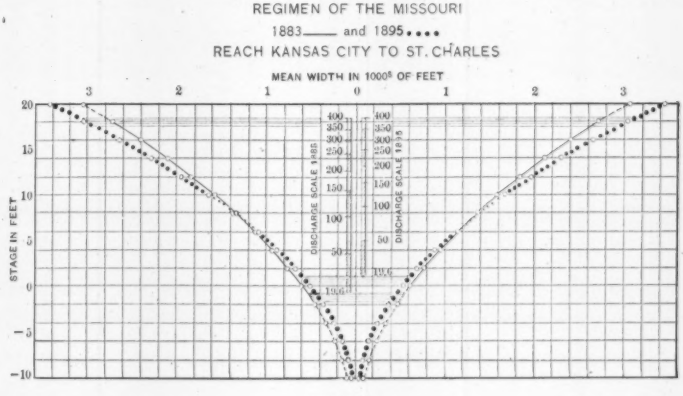


Fig. 15.

should not perhaps be taken as more than illustrations. Thus they are not definite enough to enable one to say with confidence that the extreme range between erosion and filling over this reach is a variation in mean width of some 640 ft. at high water and some 184 ft. at low water. But they are definite enough to indicate that they are something like this, and that they are certainly complementary and always opposite actions of the same system of forces.

This application of Equation II to the determination of the regimen of the Missouri is simply given as an instance of its uses. It is seen that the whole meaning of the term is changed by it; in the place of regimen being but a convenient expression for a sequence of unknown phenomena with no known relations between them, it becomes the re-

cognized resultant of the counter-actions of erosion and bar building, determined accurately and calculated easily.

Of course nothing is calculated easily from data which are not adapted to it, and the laborious processes here followed should never have to be repeated. Continuous automatic-gauge records, referred to the proper plane, should read the flood movement directly between their traces of stage, and give the values of m with precision from tributary to tributary, and through reaches which have no great changes of flood forms. The longitudinal sequence of the equal stages in these reaches is also easier to get than high and low-water slope lines, and, once determined, its mean value for the reach is probably much more nearly a constant. With these then it is simply a matter of taking groups, each of some six or eight discharge observations, at high-water, at mid-stage and at low-water, to determine positively the scales of stage at one or more locations in a long river. And, transferring these from gauge to gauge, the whole river is then known absolutely, and its form and variations of form can be calculated.

It is this which is offered, in the place of the endless soundings of cross-sections, for the study of the effect of work in the construction reaches, levee effects or reservoir effects, and for all the sequence of changing regimens in alluvial rivers, which perhaps no amount of time and money spent in hydrographic surveys would ever even detect.

It is hardly necessary to say that such data may open new problems in river hydraulics; this method only promises to answer those in sight; and even for those more experiment and more work than is shown herein may possibly be needed. But it is worth understanding clearly that the value of $\frac{dQ}{dh}$ and m for every point of stage is the equivalent of a water-line survey of the reach at that stage. True, the actual measures of distance in the survey have a smaller percentage of error than can be looked for in these calculated results, but measuring up an irregular surface area from the field measures has not. The real defect in the survey, however, is that it can never catch the reach at any one stage, and to reduce it to a uniform stage is difficult, to say the least of it. On the other hand, the flood movement not only takes the areas on planes of equal stage, but, in its sequence from high to low water, readily integrates them into the whole river prism.

Of course, with the regimen given, Equation II may be used to cal-

culate the discharge; but in these changing regimens there is always the corresponding uncertainty in a determination of the absolute value of discharge from gauge readings. No surface levels of any kind in any channel can give its discharge unless some element of the size of the channel is given at the same time, and, in rivers like the Missouri, the first thing is certainly to understand its changes of regimen.

But there are other rivers whose regimens may be fixed, or at least which change so little that it would be enough to check them at long intervals or ignore their changes altogether, and in these, when the regimen of a reach had been once established, the flood movement through it would give a continuous record of its discharge, in all its conditions of rise or fall, probably as accurately as if its waters had been run through some of the best-studied forms of weirs. And if the drainage system of a continent is ever to be taken, and such of its elements determined as can be put together consistently, it would certainly seem that some such method of measuring it up and keeping its record would be absolutely necessary.

DISCUSSION.

Mr. Todd. A. MILLER TODD, Jun. Am. Soc. C. E. (by letter).—The author has contributed a valuable addition to the literature on hydraulics and on the subject of Mississippi River Improvement.

While the principles involved are not by any means new, the method of deduction, and of the transfer of the discharge-gauge relation from gauge to gauge is altogether so, as far as the writer knows.

The results obtained present at once, in convenient form, a discharge-gauge relation for each gauge considered, which is most interesting and useful to river engineers, especially those in charge of levees; for, if the relation is demonstrated to be reliable, it furnishes a clue to possible gauge heights when the river is confined, which, for any very great flood year, has never been done. In other words, say that we know the maximum discharge, due to pass a given place; to what height on the gauge will that discharge raise the surface level of the water?

The question of possible gauge height is a paramount one to the levee engineer when it becomes necessary to establish grades for his levees, when the river is confined throughout its length, or any portion of its length.

While the paper may not give conclusive evidence on the subject, yet we may apply the discharge-gauge relations, and compare results with those obtained by different methods of computation; which methods, in every case, are crude and more or less hypothetical. We may thus check former calculations, or inaugurate further investigations and calculations, in order to reconcile the discrepancy if any is found to exist.

It is very true that it is difficult and expensive to measure the discharge at a sufficient number of points, and to study very closely the progress and attendant phenomena and laws of water in its passage through great channels like the Mississippi below Cairo. Observations which have been made seem discordant and discrepant, and methods have been crude, but, beyond a doubt, work which has been done along this line cannot be praised too much. From the very first discharge measurements taken in the Mississippi, the methods have been improved upon, and the consequent results enhanced in value, up to the present time; and the writer thinks this improvement, both in methods and results, will continue. Therefore, he would not too hastily declare the author's method of studying river phenomena one which could supersede the old altogether; but, as before stated, the author's principles, properly applied, should furnish a valuable check and help to river engineers in future studies and investigations.

Mr. Todd.

The author gives what he calls (average) "discharge scales." The writer has at hand a "Monograph" by the author "on Reservoirs and their Effects on the Floods of the Mississippi System" in which these "scales" * are plotted to a much larger scale, and they are very interesting to study. The writer has tested them with observed discharges taken at various times, and has found that the scales agree very closely with the actual results obtained from field discharge observations, and, in all instances and at all stages, they seem to show the average discharge.

In a number of lines of study, these average scales, as they are, should give correct results. But it occurs to the writer that, in many instances, before the levee engineer can accept the plotted values as correct, he must take into account the local condition of the water surface in the reach he has under consideration; whether it is rising, stationary or falling, and the rate of rise or fall; these are conditions which affect the flow. The author calls attention to the fact that, "the same change in the conditions of flow which would change the surface level by 1 ft. in a river from 50 to 100 ft. deep, would only change it by 0.1 ft. in a river from 5 to 10 ft. deep." So that, while the difference in discharge, due to whether the reach is rising or falling, is small at low stages, it becomes, at high stages, of such magnitude that it cannot be neglected. The author states that the time occupied by a change of level, passing through a given reach, is absolutely constant; but the writer understands that most, if not all, river engineers hold, from observation, that a fall travels more slowly than a rise; and, even at low stages, a rise in the reach, if the latter is long enough, will overtake a fall. Thus m in the author's formula cannot strictly be reckoned as a constant.

The author calls attention to a change of plane at Arkansas City, and offers possible explanations as to the cause, neglecting entirely what seems to be the true cause of the change; at any rate, as eminent an engineer as William Starling, M. Am. Soc. C. E., who is a recognized authority on river subjects, accounts for the identical change, and plots the identical figure used by the author, Fig. 7. In a paper by Major Starling,† Fig. 24, on page 450, shows, evidently, that the change of plane is that which recurs regularly, shifting from one to the other, according to whether the river is rising, stationary or falling.

In that valuable paper Major Starling undertakes to foretell the probable gauge height of maximum flood discharge in reaches where the floods have never been confined. He frequently calls attention to, and takes into account, the difference in discharge, at equal stages,

* House Document, No. 141; 2d Session, 55th Congress, Plates 8 and 9.

† "The Discharge of the Mississippi River," *Transactions*, Am. Soc. C. E., Vol. xxxiv, p. 347.

Mr. Todd. due to "rising river" and "river after rise," and he always obtains two curves, calling them the "two branches."*

In Fig. 16 the writer has plotted the discharge observations referred to by both the author and Major Starling, and has fitted and marked distinctively, curves through certain periods of rise and fall.

It will be seen that the fall on February 4th had something to do with the shifting of the discharge-gauge relation up to the stage on that date, and there can be plotted, through the points indicating a fall, a distinct and separate curve from that drawn through the points marking the rising stages. This second branch is traced down as long as the river falls. A rise commences on March 4th, and, instead of the discharge-gauge relation agreeing with the rise of December 30th to January 18th, the discharge seems to pass at considerably higher gauge readings. Under the circumstances, the writer thinks the foregoing fact should be expected. The average datum area was practically the same during both rises; while, from December 30th to January 18th, 19 days, the river rose 25 ft., or an average of 1.3 ft. per day, and from March 6th to March 23d, 17 days, it rose only 13 ft., or an average of 0.7 ft. per day.

For any portion of the reach in which Arkansas City is situated, if there are gauge relations, which cannot be doubted, then, for a rise of 1.3 ft. per day, the slope is bound to be greater than for a rise of only 0.7 ft. per day. Assuming, as the author does, that the miles per day traveled by a change of gauge height in a given reach A B, Fig. 17, is constant; assuming, also, that 1 ft. rise at A is equal to the same amount of rise at B, or that the gauge relation of B to A is 1, and that the time interval is 1 day; also, suppose that, on a given day, the gauge at A shows a rise of 0.7 ft., then the full line A_1-B will represent the general slope of the river on that day; the next day there will be 0.7 ft. rise at B and also at A, and for all succeeding days, as long as the increment of 0.7 ft. daily at A obtains, the slope on those days will continue parallel to A_1-B. If a 1.3-ft. rise is recorded at A, the dashed line A_2-B will represent the general slope for that day, and for all succeeding days of the 1.3-ft. rise. Now, for example, assume the distance, A to B, equal to 60 miles, then the increase of slope of the 1.3-ft. rise over the 0.7-ft. rise is 0.006 ft. per mile. The slope being a function of discharge, the discharge-gauge relation varies accordingly.

It is the writer's opinion, that, as long as the river is rising or falling at nearly a constant rate, if true discharge measurements could be obtained and plotted, the points would lie in two perfectly regular curves, one representing the rising river, and the other the falling river, for that rate of rise or fall; and points for any other rate of rise or fall will depart from these curves more or less. So that, for

* "The Discharge of the Mississippi River," *Transactions*, Am. Soc. C. E., Vol. xxxiv, p. 465, Fig. 29.

the conditions of the water surface, with constantly varying rates of rise and fall, the actual curve of discharge-gauge relations is never a regular one. But, between an imaginary curve, representing the discharges passed throughout a rise to extreme flood height, rising at the constant rate of the greatest known average rise per day, and a curve representing a fall under the same conditions, there may be

Mr. Todd.

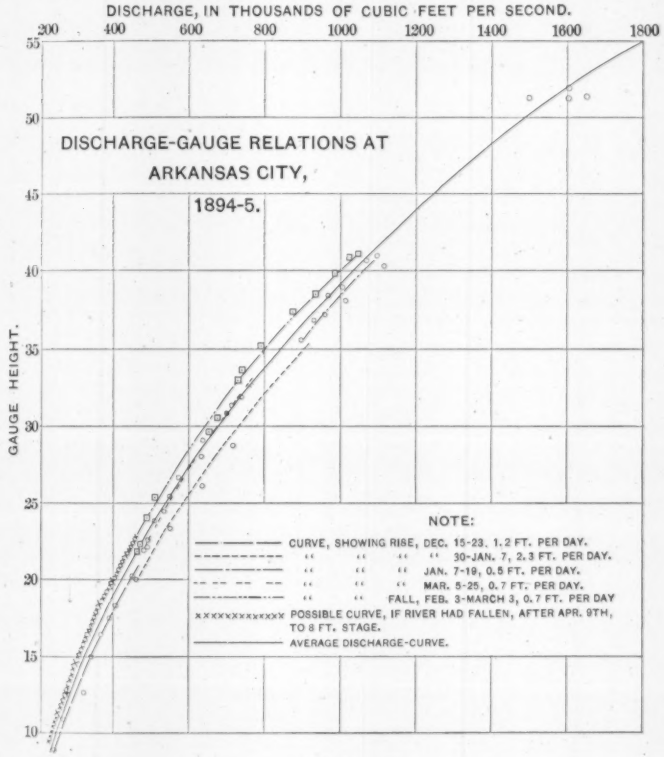


FIG. 16.

constructed a mean curve which would give a basis by which to reckon discharge-gauge relations under all varying conditions of flow.

If the observations from which Fig. 16 was derived had been continued, and the river had fallen to a stage of 8 ft., without being influenced by any sudden rise or fall, the points obtained, the writer thinks, would lie about where the x marks are indicated. Taking these points into consideration, and also the points plotted at 50 to

Mr. Todd. 51.5 ft., obtained from discharge observations of the Mississippi River Commission in 1897 and 1898,* and projecting a mean curve through all the points obtained, as indicated by the full line, this curve, in similar form, would correspond to the author's discharge scales, and is about what he would obtain for Arkansas City, if not exactly the same.

Fig. 16 shows plainly the departure of some of the observed discharges from the average discharge-curve. For investigations requiring the summation of discharge over a period covering a rise and the corresponding fall to about the same stage, the scales constructed by the author cannot be improved upon. But, as before stated, they will not do where the discharge covering any given day, or the maximum discharge, is wanted. To render the average discharge curve useful in such instances, corrections must be applied to all stations, at least below Cairo, in the Lower Mississippi; and, the writer thinks, that the correction depends principally on the slope of the surface of the water in the reach in which the gauge station under consideration is located, and at the time the discharge corresponding to a given gauge height is desired. These slopes can be had roughly, at the present time, and, if the gauge stations were located and kept, as suggested by the author, the slope between them could be ascertained accurately at all times.

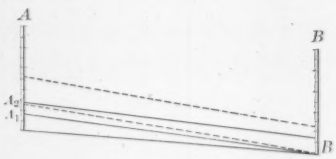


FIG. 17.

The author states that in certain instances corrections should be applied, and undoubtedly he has taken all the foregoing facts into consideration, and probably has a method of applying the correction, not hinted at by him or conceived of by the writer. Possibly the writer, in his limited study of the problem, has laid too much stress on the slope as due to the many various conditions of the river, owing to the rise and fall alone; at any rate, he hopes that the author will state, in his closure, what his corrections are, and to what extent they will affect the quantities given by the discharge-scales.

There is one thing certain, as the author states, in transferring the investigations down the river below Cairo, the problem becomes complex indeed, due, principally, to the enormous number of variations and combinations of slope and momentum possible, according to the condition of the water level, not only in the single reach, but probably in several reaches above, which may affect the discharge materially; and also to the conditions and stage existing in the various tributaries. However, the only way to arrive at any tangible results, in clearing up and solving this great problem, is to keep hammering

* See corresponding reports of the Mississippi River Commission.

away at it; and the more we hammer, the sooner the desired end will be attained. After the levees have been maintained throughout several extreme high waters, we may expect to have some light on the subject. Heretofore, all the extreme high waters, after passing Cairo, have first been restrained in one place, and then have broken the levees and inundated basins in other places, the conditions of no one year repeating themselves the next; and it is impossible to study, with any degree of satisfaction, a flood spread over 100 000 square miles and upward. One flood, that of 1898, of comparatively short duration, which came within 2 ft. of the 1897 water at Cairo, was restrained successfully where leveed. The St. Francis levees were not completed, within some 100 miles of Helena, but the remaining portion is now being constructed. This basin being for a great part still open, the question of the effect of its closure on the flood plane, or gauge at Helena, is still problematical. Mr. Todd.

So that, for many years to come, the subject presented by the author will be an important study to all engineers interested in the Mississippi River problem, and it is to be hoped that he will give us, at some future date, the results of his further investigations.

GEORGE W. RAFTER, M. Am. Soc. C. E. (by letter).—This paper is an admirable illustration of the present tendency in hydraulics, not to attempt to express complex relations drawn from a large number of cases by a single formula, but rather to work out each case by itself, on its merits. The recognition that the discharge of a stream for a given gauge height will be different for rising from what it is for falling stage, with each in some proportion to rise and fall, is a case in point, as is also the discussion of the effect of "bed in train." Frequently, the tendency has been either to include all such phenomena in a single expression, or to ignore them entirely. The best illustration, however, is found in the conclusion of the paper, that there is a particular equation which expresses the hydraulic relations of rivers better than the formulas in common use, but which does not in any degree apply to pipes, conduits and uniform reaches of straight channel. Mr. Rafter.

While the paper thus illustrates a desirable improvement in hydraulic studies, it contains, further, a series of generalizations which assist one materially in comprehending the complex series of physical facts entering into the flow of a large stream, where bends, irregular bed and other disturbing influences tend to complicate the phenomena. For all such, notwithstanding current practice, it is well to recognize that $v = c\sqrt{rs}$ can have at best only casual application, and as a demonstration of this point Mr. Seddon's paper can hardly be excelled. But for straight reaches of artifical channel with uniform cross-section, the conditions of flow are so different from those of meandering and silt-bearing streams, that deductions applicable to one may not apply

Mr. Rafter. in any degree to the other. For such a channel, the theory of velocity-slope relations becomes, the same as for pipes and conduits, all important. As shown by the author, the views expressed in the paper do not apply to these cases, but are to be considered as confined to large streams with relatively flat slopes. Nor, so far as the writer can now see, will they apply to small streams and mountain torrents, for both of which the Chezy formula is more nearly applicable. This, however, is merely in line with the proposition to, so far as possible, work out formulas suited to each specific case.

Mr. Le Conte. L. J. Le Conte, M. Am. Soc. C. E. (by letter).—It has been generally known that there exists a large amount of valuable information on the files of the several district offices in the Mississippi Valley which, from lack of funds, has never been properly studied and digested by any competent person. The author has undertaken this task, and has presented many facts and important conclusions.

He brings out strongly the important feature that local slope is of no importance in the formulas for stream flow, because it is an unstable as well as a secondary result of local changes in cross-section; and since the cross-sections themselves are constantly changing, the local slope also changes correspondingly, and yet the discharge remains constant all along the entire reach. These potent facts show how careful an engineer should be in selecting the upper and lower limits of the characteristic reach, in order to ascertain, for the entire reach, the true average fall, the true mean characteristic depth, and the corresponding constant discharge. That is to say, he must not allow himself to be misled by any non-characteristic local slope or cross-section, their lack of stability being in itself sufficient evidence of their secondary character. As an extreme case, showing the lack of influence of surface-slope, we have at Donaldsonville an actual reverse-slope extending up the river for 20 miles or more, and yet the great quantity of water moves down the river with the same discharge as where the slope is down stream.

The author's system of triple gauges at each station is certainly the most rational device, and, at the same time, gives the best and quickest results. The amount of useful information which can be obtained by means of such simple apparatus is remarkable.

The author's comments on levee effects, in always lowering the low-water plane and thus facilitating flood-water propagations down the river, will be welcome news to river engineers as well as riparian land owners. It is a matter of paramount importance in reclamation works, and should command the closest attention. The whole story is shown in an indisputable form by a study of the discharge scales for the past ten years.

The author's statement of the facts pertaining to flood movement and his mathematical deductions therefrom are certainly interesting

and valuable. The full value of their bearing upon the actual flattening out of the flood waves and the correspondingly rapid flood-propagation down the river channel are matters of great importance when dealing with large rivers, and the writer thinks that, at least in the lower divisions, even of rivers of ordinary size, the influence of this flood-pulse has been largely underestimated. The author emphasizes the fact that in the Lower Mississippi the prodigious flood-movement cannot be explained by the direct flow of the waters down stream, because the flood-wave crest travels down the river four times as fast as the water. This feature, together with the mathematical deductions, shows unmistakably that a close relation exists between these flood-propagation phenomena and the laws governing tidal propagation in shallow estuaries and tidal rivers; the only difference being that the flood-pulse originates in the upper river and is propagated down stream, while the ordinary tidal-pulse originates in the ocean and is propagated up stream against the flow of the current. Mr. Le Conte.

It is to be hoped that the author will continue to give us information in this line and thus enable us to weed out the weak points in existing standard formulas based on experiments of small scale.

JAMES A. SEDDON, M. Am. Soc. C. E. (by letter).—From the limited discussion offered, the writer has come to question whether he really has succeeded in making the hydraulic system, presented in his paper, as clear as he thought he had. As the field of river data has in it certainly a good deal of brush wood, in which even the trained investigator may lose himself utterly for a time, and which probably deters many from ever entering it, the writer has concluded that he can best serve the purposes of a discussion by presenting briefly the outline of his system in its application to a simpler case, and one with which engineers are more generally familiar. Mr. Seddon.

Assuming for this a flume, say 5 ft. wide, set on some uniform grade, which it is not now necessary to consider; and taking 10 000 ft. of it as the reach through which observations are to be made; automatic gauges with zeros at the bottom of the flume set at the upper and lower end of it; the water run out and the whole thing ready for the experiment.

The flow then would be started, very little at first, increasing steadily until the flume was running full for a time, and then gradually shut off and the water again drained out of it. During this time the records of the two gauges would be the only data required to determine the discharge of the flume at any and every level and through all these variations of slope found in its filling and emptying.

For instance, each automatic gauge would trace the changing level to a common time scale, and should, of course, be set accurately to give this from the bottom of the flume, enlarged if necessary, and so proportioned to the time scale as to mark the coincident time and level

Mr. Seddon. most distinctly. Then, say, from the trace of the rising level at each gauge, the data of Columns (1), (2) and (3) in Table No. 2 are taken.

TABLE No. 2.

Water level in flume h, in feet.	Time at which Level is Reached on Gauges.		Difference of time, in seconds. $= \Delta T$.	$\frac{10\,000}{\Delta T} = m$.	$m \times 5 = m W$.	Discharge, in cubic feet per second $= Q$.
	Upper gauge.	Lower gauge.				
(1)	(2)	(3)	(4)	(5)	(6)	(7)
0.1	6 00 00 A.M.	8 34 20 A.M.	9 260	1.08	5.4	0.5
0.3	6 40 00 "	8 54 30 "	8 070	1.24	6.2	1.7
0.5	7 20 00 "	9 19 00 "	7 140	1.40	7.0	3.0
1.0	8 10 00 "	9 42 40 "	5 566	1.80	9.0	7.0
1.5	9 00 00 "	10 15 50 "	4 550	2.20	11.0	12.0
2.0	10 00 00 "	11 04 10 "	3 850	2.60	13.0	18.0
3.0	12 00 00 M.	12 49 00 P.M.	2 940	3.40	17.0	33.0
4.0	2 00 00 P.M.	2 39 40 "	2 380	4.20	21.0	52.0
5.0	4 00 00 "	4 33 20 "	2 000	5.00	25.0	75.0

From this, Columns (4), (5) and (6) are readily computed; and, platting these values of $m\ W$ to h gives the line shown on Fig. 18, which is expressed by the empiric equation $m\ W = 5 + 4\ h$.

Then from Equation II,

$$\frac{d\ Q}{d\ h} = m\ W = 5 + 4\ h,$$

and integrating,

$$Q = 5\ h + 2\ h^2 \pm c.$$

In the more general case, where h is the stage scale and is taken from a zero at an arbitrary low water, c, the constant of integration, is simply the regular discharge of the river at that level, upon which the variations of discharge given by the flood movement are imposed. But here, where h is taken from the bottom of the flume, and the values of Q and h become zero together, c is then zero, and the complete discharge equation is $Q = 5\ h + 2\ h^2$, from which the values of Q given in Column (7) are computed, and platted on Fig. 18 show the discharge curve as determined.

This, of course, is merely an illustration, and may not correspond with any actual case; and, indeed, no case is actually fixed until the grade of the flume is given, which has not yet been even considered. It simply presents the process of determining, in a channel of given dimensions, a whole discharge sequence at once merely with the records of two gauges.

Neither does the writer claim that it is a better measure of the discharge here than the ordinary method of observing the velocity at a number of points in the section and from these computing the flow; or the alternative of running a steady flow into a basin of a given size,

and from the filling in the period calculating the discharge. Considering the wide range of discharge values which the proposed method covers in a single operation, with no meters to rate or observers' errors to question, and the whole thing reduced with little more work than that of a single discharge observation, if the writer had any experiments on the flow of conduits or flumes to make, he would certainly try it. But, until it is tried, he is not ready to consider its precision. All this rests upon the precision with which the time element involved in m may be determined, and is a matter yet to be tested. Mr. Seddon.

But this is not an untried field in the writer's studies of rivers. He has shown that here in favorable cases m may be determined within limits hardly exceeding 1%, and even on the Missouri probably close to 3 per cent. This has been done also simply with the ordinary gauge data, and how unsuitable this is for such determinations should be understood. In the first place, the gauge observers are local men employed in other occupations and paid a small sum monthly to take the readings and send them to the different offices, and there is no guarantee that they do not generally read their gauges earlier or later than the set time, by an hour or so, as suits their convenience. Again, the water surface is frequently quite rough, and even the most careful man may make a mistake of a tenth or more in estimating its level, with waves running a foot high. And, finally, the gauges themselves are often set with no nice regard for any real precision in their readings, taking in the pulse of an eddy, or the extreme variations of slope through a bridge span, and in the case of cable gauges, all the temperature errors incident to measuring down from the lower cord of a bridge to the water surface with a weight at the end of a wire.

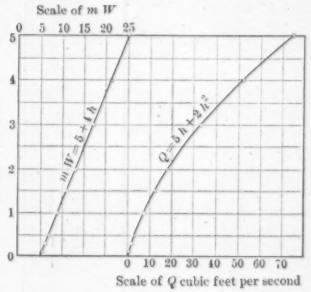


FIG. 18.

The precision attained in the determination of m, with such data, leaves no doubt whatever that surface observations, planned and taken with that specially in view, would bring this proposed measure of discharge into a field of more reliable and exact data than the best that has yet been taken, not to speak of the whole mass collected, which now requires years of study to sift in order to form some idea of what is reliable in it.

This, however, is simply the case where the dimensions of the channel are given. In tidal rivers, for the determination of their peculiar form of discharge curves, with a zero at both the upper and

Mr. Seddon. lower levels joined by plus and minus branches, probably this process would be all that was wanted. But in other rivers it may be desirable to first calculate dimensions; and, going back to the flume to consider such a case, it is plain that the discharge curve of Fig. 18 is not the curve for the head of the reach nor the foot of it, but for some intermediate average; and that this again changes with the grade of the flume and different rates of filling and emptying.

The only constant in the matter, with a set grade, is the intermediate value of discharge between equal rates of rise and fall, or the uniform flow at the different levels, and this corresponds to the mean of the rising and falling flood movements. In the case of the flume, very possibly, accurate dimensions might be determined with simply rising discharges observed in the middle of the reach and the coincident flood movements through it; but in the river, where a general stage scale for the reach is also to be gotten, the process of standard curves and mean m^s is preferable.

However, the dimensions once determined in this way, if the form of the river is fixed, W at any time, in the equation $\frac{dQ}{dh} = mW$, is given, and simply measuring m then will settle all questions of what effects the larger slope on the rise and the less on the fall may have in that reach. So far as his own studies go, with these rivers of fixed regimen, the writer would say that one reach may show these effects and the next may not; and that this depends on whether a general slope controls the flow there or special sections. But it is certainly a field that needs further observations.

In a general way, also, the same may be said of alluvial rivers. But since the writer in 1885 turned a given flow down an inclined plane covered with river sand, and found that it came to about the same velocity no matter what inclination he gave it, he has given up trying to explain all the phenomena of alluvial river flow with velocity-slope relations. And, indeed, he thinks it is well here for every investigator to first try and see broadly what slope does not explain before he puts any great amount of time into fitting local data thereto.

Take a case from the Arkansas City, 1884–85, discharges illustrating it with Fig. 19, in which scales are ignored for the sake of bringing the matter out clearly. It is seen that this change of plane, or different levels of the same discharge in the first and second periods, comes in between Helena and Arkansas City and from there on is continuous to Vicksburg. If the different slopes on the rise and fall are taken to explain it from Arkansas City down, it must at the same time be explained how this flood with about the same slope differences shows no such marked change in the discharge levels from Cairo to Helena. And, further, in the same connection, while the writer does not draw conclusions from the data of single sections, it may yet be noted,

that Mr. Starling's coefficients* for this Arkansas City section, with these slope differences taken into account, show an even more marked difference between the two periods than that given by the discharges. Mr. Seddon.

The writer calls this a change of plane, and, following the flood from Cairo down, as he finds no general difference of any such magnitude that he can assign to the varying slopes of the rising and falling river, he does not recognize them as the cause of it. He does not mean that such effects are not there, but simply that they are not in this proportion. But these are not the only changes of slope involved in the matter. In Fig. 19 a change in the slope of the discharge levels from Helena is shown, and it has been stated that this corresponds roughly with a flood out of the Arkansas. It should also be noted that in such a period when the discharge is 600 at Helena it may be 700 to 800 at Arkansas City; and this change in the discharge level is then but a fraction of the actual change in the surface slopes leading down to it. Had Mr. Starling, in place of taking his slopes from Arkansas City to Greenville, taken them from above to the mouth of White River, where this change really comes in, he would have found a much better ground on which to base these discharge differences.

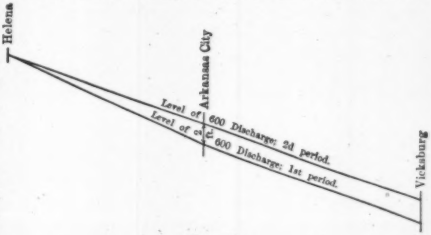


FIG. 19.

But why, when such a change has once come into the river, it seems to hold for a considerable time after the tributary flood has run out, and is continuous, apparently without change in its magnitude, for hundreds of miles down, are matters which the slopes do not explain. However, to answer Mr. Todd's question as to what the writer's system of corrections is: He will have to call them, for the present, simply accumulated experience of where and under what conditions such changes are likely to come in, and their probable magnitudes. He does not feel that he has quite gotten to the bottom of this phenomenon, and is not ready to say that any single cause, that is altogether satisfactory, can be assigned to it.

Going back, however, to the flume for comparison, it may be seen that while the general grade in the alluvial river is also fixed; it is still the case where at any location the bottom, or the zero of discharge, from time to time may be shifted. Simply shifting the discharge curve as a whole up or down, the writer has found covers at least the

* "The Discharge of the Mississippi River," *Transactions*, Am. Soc. C. E., Vol. xxxiv, Fig. 20.

Mr. Seddon. greater part of the phenomena; and as this is done in a minute, and records definitely the change in both its extension of time and length of river, it suits his method of first collecting facts on which to build his theories. It, of course, makes the problems more complicated, but he has indicated the general methods of meeting them, and that is as much as can be expected in a new field such as this is. That where such changes occur, the slope is altered, is evident, but the writer takes these changes of slope as effects; the field in which slope is a primary cause he takes to be quite a different matter.

In that lies the explanation of all the general forms or types shown in the different regimens of different rivers, and the writer does not come to it in the paper presented; he has carried the subject no further than the case of the flume, where the grade was not even considered. He offers, however, a system in which these regimens may be determined, and, indeed, which promises to bring rivers into a much more exact field of calculation than that found in any other line of hydraulics; certainly the more precise determination of m is a subject for further experiment, and the application of the equation $\frac{d\,Q}{d\,h} = m\,W$ must be varied to suit the cases. But taking the given regimen of the Missouri River from Kansas City to St. Charles as an example, perhaps something of the dynamics, in which this, as a whole, stands as an equilibrium, may best illustrate his view of it.

First, the energy of the discharge and fall in this reach varies from about 750 000 H.-P. at low water to about 15 000 000 H.-P. at high water. This, then, tears down on an average about 120 000 000 cu. yds. of bank annually, or an amount which, dumped in year after year, is enough to fill this given prism up solid to some 12 ft. above low water in the twenty years in which the Government has been working on it. That it has not changed it perceptibly, makes the fact very plain that it has simply been built back again on banks and bars in other places.

Of course, a considerable part of the observed sediment in the water lies simply in this action, and for the reach from Kansas City to St. Charles, as a whole, may be said neither to come into it nor to go out of it. But taking this in about the proportion of the sediment that will settle from the water with the least check to its velocity, and which can hardly have come from distant head waters, leaves the true sediment for the year about 130 000 000 cu. yds. carried in suspension. And certainly the percentage of this which may be dropped in the reach is an additional load on the bar-building forces.

But a good part of this matter carried in suspension will not settle in a reservoir after standing 24 hours, while, even as a whole, it is but little more than the erosion on this 333 miles of the river, and, for the total alluvial stretch of the Missouri, is a minor fraction in comparison with it. Indeed, the fact of the almost uniform grade on which this

river is set leaves little doubt that it has been long ago leveled up to bring the sediment entering it into practically a thorough transportation; but, in any event, in all such cases the probable proportions are such, that if no changes from erosion are shown in the 10 or 20-year periods, about 1 000 years is the least time in which material effects may be looked for from the matter carried in suspension. Mr. Seddon.

Of course, all this only shows when the long reach of river is taken into view altogether. At a given location the erosion may be small, and the bar building large, while the sediment carried through in suspension is so great in proportion to both of them that anything seems to be possible from it. It is not until the engineer gets away from actually looking at it, and sees, in his mind, the hundred miles of river as a whole, as he sees the orbit of the planet, that he begins to find himself in the presence of the real dynamics of rivers, and can fairly value the processes given, in which these equilibriums may be measured, and their variations traced from river to river, and from season to season; a matter that at first looks as hopeless to him as chaining the distance to the moon, but which, after all, is very easily measured when we know how.

Vol. XLIII. JUNE, 1900.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

No. 872.

THE ALBANY WATER FILTRATION PLANT.

By ALLEN HAZEN, Assoc. M. Am. Soc. C. E.

PRESENTED JANUARY 3D, 1900.

WITH DISCUSSION.

HISTORICAL.

Albany, N. Y., was originally supplied with water by gravity from certain reservoirs on small streams west and north of the city. In time, with increasing consumption, the supply obtained from these sources became inadequate, and an additional supply from the Hudson River was introduced. The water was obtained from the river through a tunnel under the Erie Basin, and a pumping station was erected in Quackenbush Street to pump it to reservoirs, one of which served also as the distributing point for one of the gravity supplies. The intake, which was used first in 1873, drew water from the river opposite the heart of the city. In recent years, the amount of water drawn from this source has greatly exceeded that obtained from the gravity sources.

Some of the city sewers enter the river above the intake, but most of them are below it. In times of flood, the water thus obtained was polluted by the sewage of only a few of the city sewers. At low-water stages, however, owing to the tidal currents, the water con-

tained much sewage, which was carried up stream to the intake, and the sewage of the city was thus present, in very considerable amount, in its own water supply.

In addition to the local sources of pollution, the river received the sewage of Troy and the surrounding cities, 7 or 8 miles above, and the sewage of Schenectady, Utica, Rome, and many other places farther away.

Under these conditions the typhoid fever death rate in Albany was excessive. Professor W. P. Mason, of Troy, made a report upon the quality of the water to the Water Board in 1885, in which he stated in unmistakable terms that the water as then used was a source of disease, and should be abandoned at the earliest practicable date.

Following this, an attempt was made to secure a ground-water supply, but without results. Studies were then made for gravity sources of supply, mentioned in the reports of the Board of Water Commissioners for the years 1891 to 1893.

Failing to secure the necessary legislation to introduce a gravity supply, the Board, in 1896, investigated methods of purifying the present supply. The matter was studied by the Board and by its Superintendent, George I. Bailey, M. Am. Soc. C. E., and the writer was engaged, in January, 1897, to examine the studies which had been made, and to report upon the projects presented. This report, presented to the Board in February, 1897, recommended the general scheme previously outlined by the Superintendent, namely, to abandon the present intake, and to establish a new one at a point about two miles farther up the river, at a point above all the local sources of pollution, and to pump the water by low-lift pumps to a settling basin, from which it would flow to sand filters, and thence through a pure-water conduit to the present pumping station in Quackenbush Street.

Among other considerations, taken into account in determining upon this site, was the fact that the present high-service reservoir is not at a sufficient elevation to supply properly the highest parts of the city, although it is on the highest ground in its immediate neighborhood. It will be necessary ultimately to construct a new high-service reservoir, and the most available location for it is upon land about $1\frac{1}{2}$ miles northeast of the present Prospect Hill Reservoir, where there is an excellent site directly back of that recommended for the filters; so that when it becomes necessary to construct the reservoir,

the most advantageous location for a pumping station to supply it will be close to the filters.

The report was accepted, the recommendations adopted, and the necessary funds were provided; and, in July, 1897, the preparation of plans was begun.

As the pure-water conduit was to be placed in the Erie Canal, and as the work of constructing it necessarily had to be done during the season of closed navigation, plans for this part of the work were prepared first. The contract was let to Messrs. Hilt, Johnson, Fitzgerald & Mulderry in November, 1897, and that part of the work under the canal was completed before navigation opened, May 1st, 1898.

Plans for the filters and sedimentation basin were completed in January, 1898, and the contract was awarded in February to Messrs. T. Henry Dumary and the Wilson and Baillie Manufacturing Company, who commenced work under their contract in April; but, owing to various reasons connected with the installation of a very elaborate contractors' plant, work was not pushed actively by them until August, 1898. Contracts for the pumping machinery, and for the pumping station and intake, were let to the Prindle Pump Company and to Stone & Thurston, respectively, in June and August, 1898, and the work was carried out during the fall and winter. Gates and all special valves were furnished by the Eddy Valve Company. The work was sufficiently advanced so that a part of the plant was put in operation on July 27th, 1899. The old intake was closed on September 6th, 1899, since which time no unfiltered river water has been pumped to the city.

Source of Supply.

The Hudson River, at the point of intake, has a drainage area of 8 240 square miles. Of this, 4 570 square miles are tributary to the Hudson above Troy, 3 502 are tributary to the Mohawk, and 168 are tributary to the Hudson below the Mohawk.

The average annual flow of the streams probably amounts to at least 1 000 000 galls. per square mile per day, or over 8 000 000 000 galls. per day. The minimum flow is only a small fraction of this amount. No gaugings of the river at this point are available. George W. Rafter, M. Am. Soc. C. E., has made a study* of the flow of the Upper Hudson, and estimates the minimum flow at Mechanicsville at

* Report of the State Engineer and Surveyor for 1895, p. 119.

0.24 cu. ft. per second per square mile of tributary drainage area. Assuming that this figure applies to the whole of the Hudson above Troy, and taking a somewhat lower figure, namely, 0.15 cu. ft. per second per square mile, for the discharge of the Mohawk and of the Hud-

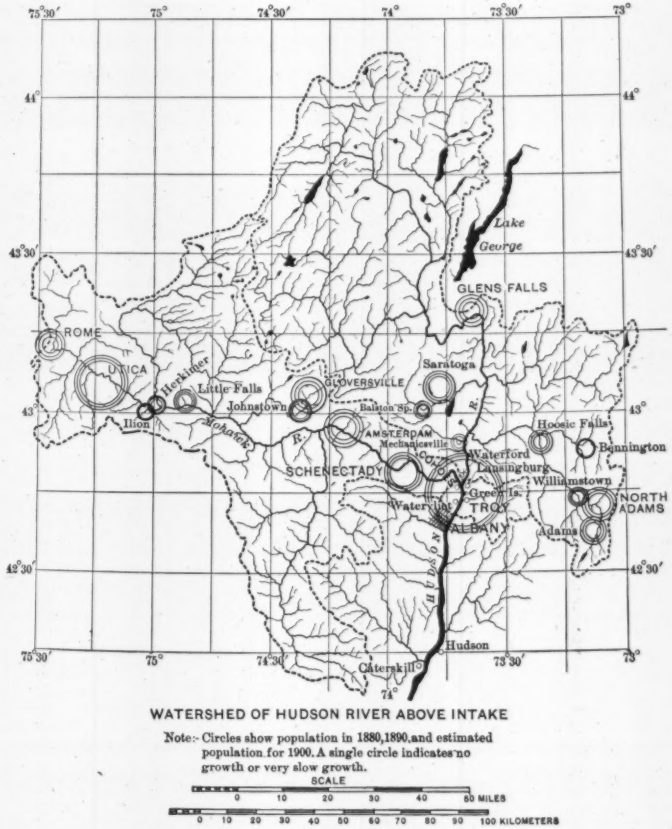


FIG. 1.

son below Troy, we arrive at a minimum flow of the Hudson at Albany of 1 647 cu. ft. per second, or 1 060 000 000 galls. per 24 hours, being, in round numbers, 100 times the average amount of water now taken from the river for water-works purposes, and at least 50 times the maximum.

Pollution of the Raw Water.—Table No. 1 gives the names of the cities and larger towns upon the river above the intake, with estimated populations and distances. The largest of these places are also shown upon the map of the water-shed, Fig. 1.

TABLE No. 1.—Cities, Towns and Villages on the Water-Shed of the Hudson River above Albany, with Populations of 1 000 and Over.

Place.	County.	Approximate distance above intake. Miles.	Population in:		
			1880.	1890.	1900. (Estimated.)
Troy	Rensselaer	4	56 747	60 956	65 470
Watervliet	Albany	4	8 820	12 967	19 040
Green Island	Albany	5	4 160	4 463	4 788
Cohoes	Albany	8	19 416	22 509	26 450
Lansingburg	Rensselaer	8	7 432	10 550	14 980
Waterford	Saratoga	9	(1 822)	1 822	(1 822)
Mechanicsville	Saratoga	19	1 265	2 679	5 358
Schaghticoke	Rensselaer	27	(1 258)	1 258	(1 258)
Schenectady	Schenectady	28	13 655	19 002	26 450
Schuylerville	Saratoga	32	1 617	1 387	1 190
Greenwich	Washington	39	1 231	1 668	2 247
Fort Edward	Washington	43	4 680	4 424	4 182
Hoosic Falls	Rensselaer	44	4 530	7 014	10 860
Amsterdam	Montgomery	44	9 466	17 336	31 730
Cambridge	Washington	45	1 482	1 598	1 723
Sandy Hill	Washington	46	2 487	2 895	3 371
Glens Falls	Warren	49	4 900	9 509	18 450
South Glens Falls	Saratoga	49	1 083	1 606	2 387
Saratoga Springs	Saratoga	51	8 421	11 975	17 010
Ballston Springs	Saratoga	51	3 011	3 527	4 131
Fultonville	Montgomery	54	881	1 122	1 429
Fonda	Montgomery	54	944	1 190	1 500
Johnstown	Fulton	56	5 013	7 768	12 040
Bennington, Vt.	Bennington	56	(3 971)	3 971	(3 971)
Gloversville	Fulton	58	7 133	13 864	26 930
Canajoharie	Montgomery	63	2 013	2 089	2 168
Williamstown, Mass.	Berkshire	63	3 394	4 221	5 250
Corinth	Saratoga	64	510	1 222	2 444
North Adams, Mass.	Berkshire	68	10 191	16 074	25 340
Luzerne	Warren	68	468	868	1 610
Schoharie	Schoharie	68	1 188	1 028	889
Fort Plain	Montgomery	69	2 443	2 864	3 358
Middleburg	Schoharie	73	1 123	1 139	1 155
St. Johnsville	Montgomery	74	1 072	1 263	1 488
Cobleskill	Schoharie	74	1 222	1 822	2 717
Adams, Mass.	Berkshire	75	5 591	9 213	15 181
Warrensburg	Warren	82	748	893	1 073
Little Falls	Herkimer	82	6 910	8 783	11 160
Herkimer	Herkimer	90	(1 353)	(1 353)	(1 353)
Mohawk	Herkimer	91	1 441	1 806	2 264
Ilion	Herkimer	93	3 711	4 057	4 436
Frankfort	Herkimer	95	1 085	2 291	4 582
Utica	Oneida	107	33 914	44 007	57 090
Whitesboro	Oneida	113	1 370	1 663	2 018
New York Mills	Oneida	114	(1 195)	1 195	(1 195)
New Hartford Mills	Oneida	117	710	912	1 171
Oriskany	Oneida	118	597	860	1 239
Clinton	Oneida	127	1 236	1 269	1 303
Rome	Oneida	127	12 194	14 991	18 430
Waterville	Oneida	133	(1 734)	1 734	(1 734)
Total, not including rural population			272 838	354 672	479 415
Per square mile			33	43	59
Rural population per square mile (in addition)			—	30	—

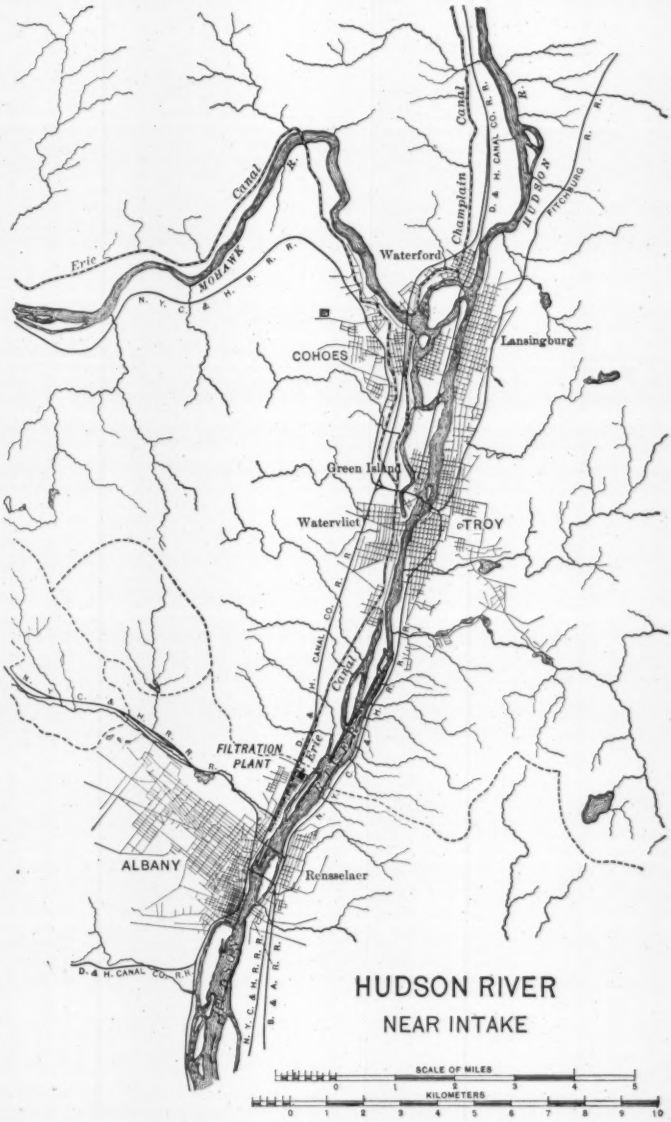

Erie Canal
Mohawk R.
N. Y. C. & H. R. R. R.
Champlain Canal
D. & H. CANAL CO. R. R.
Hudson R.
Fitchburg R. R.
Waterford
COHOES
Lansingburg
Green Island
Watervliet
TROY
D. & H. CANAL CO.
Erie Canal
FILTRATION PLANT
ALBANY
Rensselaer
B. & A. R. R.
HUDSON RIVER
NEAR INTAKE
SCALE OF MILES
0 1 2 3 4 5
KILOMETERS
0 1 2 3 4 5 6 7 8 9 10

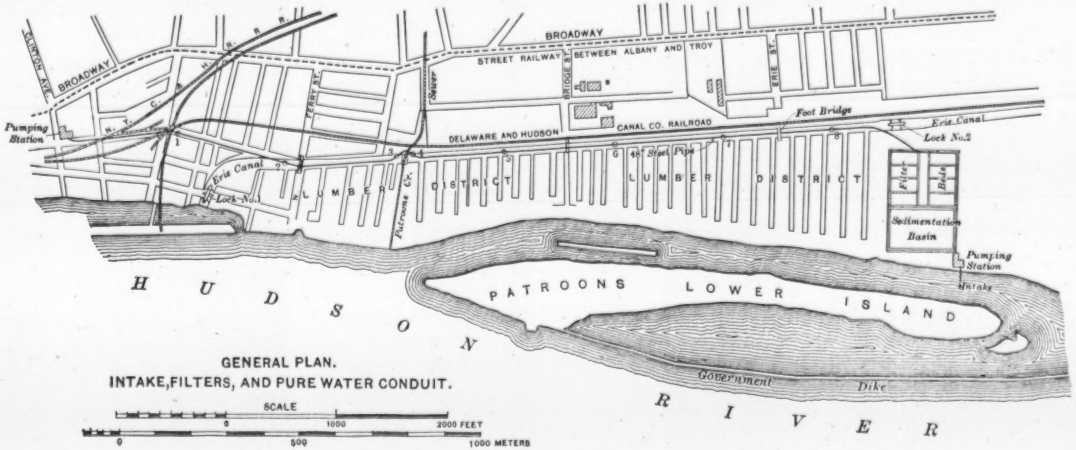
FIG. 2.

TABLE No. 2.—Bacterial Examinations of Water from Main Channel and from Back Channel at Proposed Points of Intake.

Date.	Hour.	Approximate state of tide.	Back Channel.		Main Channel.	
			Turbidity.	Bacteria.	Turbidity.	Bacteria.
1898.						
March 24..	3.00 P. M.	Low tide	.....	2 250	.....	.....
April 13..	11.00 A. M.	Rising	.....	1 927	.....	2 889
20..	7.15 A. M.	High	.....	1 970	.....	.....
20..	8.00 "	High	.....	.....	.....	2 165
June 8..	12.00 M.	High	0.04	400	.....	2 200
20..	1.30 P. M.	Falling	0.09	350	.....	825
24..	7.00 A. M.	Low	0.03	1 000	0.04	1 040
	9.20 "	Rising	0.03	760	0.04	1 120
	11.40 "	Nearly high	0.03	600	0.04	1 040
	2.00 P. M.	After high	0.03	300	0.04	4 000
	4.20 "	Falling	0.03	1 400	0.04	5 200
	6.40 "	Low	0.03	640	0.04	3 600
29..	2.00 P. M.	Nearly high	.....	.....	0.03	2 040
	2.30 "	High	0.04	860	.....	.....
July 7..	9.00 A. M.	High	.....	.....	0.03	4 360
	10.00 "	After high	0.04	560	.....	.....
13..	11.00 A. M.	Rising	.....	.....	0.03	16 780
	12.00 M.	Nearly high	0.03	17 926	.....	.....
21..	11.00 A. M.	Falling	.....	.....	0.03	13 759
	12.00 M.	Falling	0.04	18 724	.....	.....
29..	.....	.....	.....	13 759	.....	15 230
Aug. 4..	.....	.....	.....	20 200	.....	16 070
20..	.....	.....	.....	11 275	.....	7 280
27..	.....	.....	.....	17 286	.....	16 750
Sept. 2..	.....	.....	.....	12 744	.....	9 640
8..	.....	.....	.....	11 860	.....	11 289
13..	.....	.....	.....	14 700	.....	14 366
26..	.....	.....	.....	22 750	.....	17 280
Oct 1..	.....	.....	.....	19 760	.....	16 800
7..	.....	.....	.....	5 260	.....	3 724
21..	.....	.....	.....	9 700	.....	6 240
Nov. 4..	.....	.....	.....	.....	.....	3 560
9..	.....	.....	.....	.....	.....	8 760

Without entering into a detailed discussion, it may be said that the amount of sewage, with reference to the size of the river and the volume of flow, is a fraction less than that at Lawrence, Mass., where a filter plant has also been constructed, but the pollution is much greater than that of most American rivers from which municipal water supplies are taken.

Position of Intake.—The general form of the river channels near the new plant is shown in Figs. 2 and 3. Opposite the plant is a long, narrow island. The land upon which the filter plant is built fronts the back channel of the river. This back channel carried formerly a considerable proportion of the river's flow, but, to improve navigation, the United States Government has constructed a dike, the top of which is about 5 ft. above low water, and this dike cuts off most of the flow



GENERAL PLAN.
INTAKE, FILTERS, AND PURE WATER CONDUIT.

SCALE
0 1000 2000 FEET
0 500 1000 METERS

FIG. 3.

through this channel at ordinary river stages. At flood stages the dike is overtopped, and the channel takes a large amount of water.

The question arose as to whether to build an intake across the back channel and the island to the main channel of the river, or to take water from the more convenient back channel. Each point had its advantages. To determine the relative character of the water, examinations were made by Dr. George Blumer, of the Bender Hygienic Laboratory, of Albany. The most important of the results of these examinations are shown in Table No. 2.

The results, up to July 7th, are normal, and represent the condition of the water as it would be ordinarily. Afterward, commencing July 9th, the contractors dumped sand and gravel in the back channel, and took it up again by dredging, for construction purposes, with the result that this water was fouled, and the samples taken after that time do not represent its normal condition.

The results, up to July 7th, showed, in a general way, that the water in the back channel was considerably better than that in the main channel. Occasionally, there was but little difference, and this would always be the case when the river was in flood. At no time was the water in the back channel materially worse than in the main channel.

One of the city sewers enters the river at a point a short distance from the outlet of the back channel, and there was a possibility that sewage therefrom would be carried up the back channel by flood tides. On the other hand, the water in the main channel had come directly from the Troy sewers, while that in the back channel was more or less completely cut off from the main current, and was moved back and forth by the tides, and opportunities for natural purification were present in greater degree than in the main channel. These conditions, apparently, more than offset the possible admixture of fresh sewage.

The plan finally presented was to build two intakes independent of each other, one to each channel, and connected with the pumping station separately; but, in view of the superior average quality of the water in the back channel, up to the time of letting the contract for the intake, it was decided to construct at first only the intake in the back channel, and put in a stub for the other intake, the construction of which was to be deferred until such time as might seem necessary, or indefinitely, should the relative qualities of the raw water in the

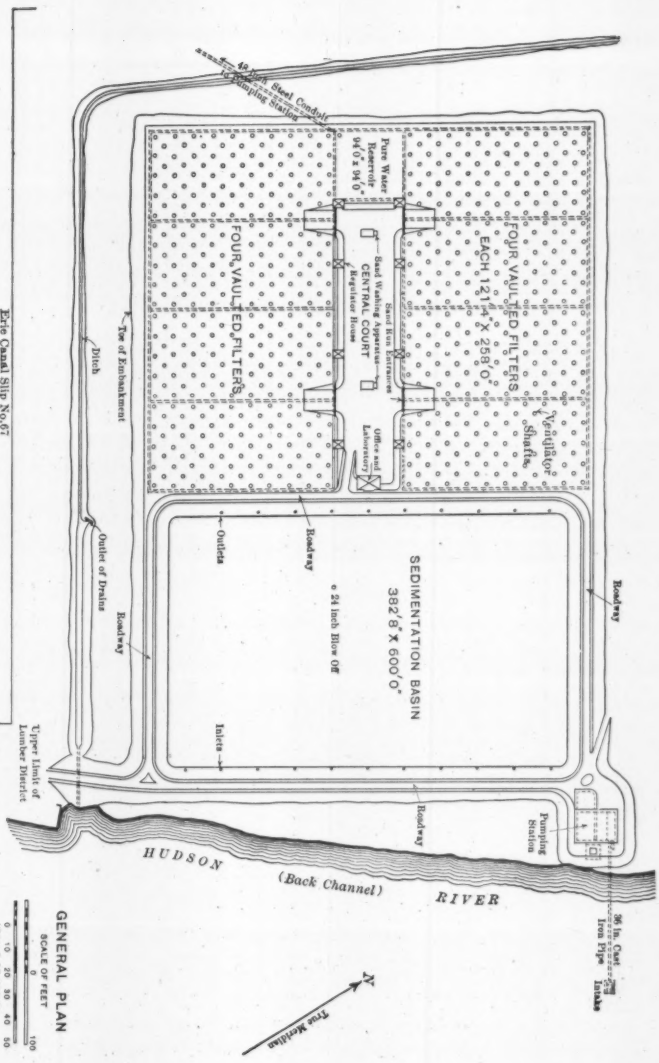

48 inch Steel Conduit
Pure Water Reservoir 94'0" x 94'0"
Sand Washing Apparatus
CENTRAL COURT
Regulator House
Sand Run Entrances
Office and Laboratory
FOUR VAULTED FILTERS EACH 121'4" X 258'0"
Ventilator Shafts
FOUR VAULTED FILTERS
Roadway
SEDIMENTATION BASIN 382'8" X 600'0"
24 inch Blow Off
Outlets
Inlets
Roadway
Pumping Station
Roadway
Toe of Embankment
Ditch
Outlet of Drains
Roadway
Upper Limit of Lumber District
Erie Canal Slip No. 67
HUDSON (Back Channel) RIVER
36 in. Cast Iron Pipe
Intake
True Meridian
N.
GENERAL PLAN
SCALE OF FEET
0 100
0 10 20 30 40 50
SCALE OF METERS

FIG. 4.

two channels remain as during the time covered by the foregoing examinations.

Description of Plant.

The filter plant, and the most important structures connected therewith, are shown in Figs. 3 to 15, and this description, for the most part, will be limited to those points which are not thus shown.

Intake.—The intake is shown by Fig. 6, and consists of a simple concrete structure in the form of a box, having an open top covered with rails 6 ins. apart, and connected below, through a 36-in. pipe, with a well in the pumping station. Before going to the pumps the water passes through a screen with bars 2 ins. apart, so arranged as to be raked readily. The rails over the intake and this screen are intended to stop matters which might obstruct the passageways of the pumps, but no attempt is made to stop fish, leaves or other floating matters which may be in the water. The arrangement, in this respect, is like that of the filter at Lawrence, Mass., where the raw water is not subjected to close screening. There is room, however, to place finer screens in the pump well, should they be found desirable.

Pumps.—The centrifugal pumps were built for the Prindle Pump Company at the Lawrence Machine Shop. They have a guaranteed capacity of 16 000 000 galls. per 24 hours against a lift of 18 ft., or 12 000 000 galls. per 24 hours against a lift of 24 ft., corresponding to a water-horse-power, in either case, of 50.5. The ordinary pumping at low water is against the higher lift, and under these conditions either pump can supply the ordinary consumption, the other pump being held in reserve. The plant is arranged, however, so that if for any reason a large quantity of water is required when only one pump can be used, water can be pumped direct to the filters against the lower head, in which case one pump will deliver a larger quantity, up to 16 000 000 galls., the full nominal capacity of the plant.

The boilers were built by James Hunter, of Albany, and are of the vertical tubular type, each of 100 H.-P. The engines are connected directly to the pumps, and were built by the Watertown Engine Company. The plant is supplied with Dean condensers, feed-water filter, and other appurtenances. The whole plant is in duplicate, either half of which is capable of supplying the ordinary consumption, or the consumption up to the limit of capacity of the filters under the above-mentioned conditions.

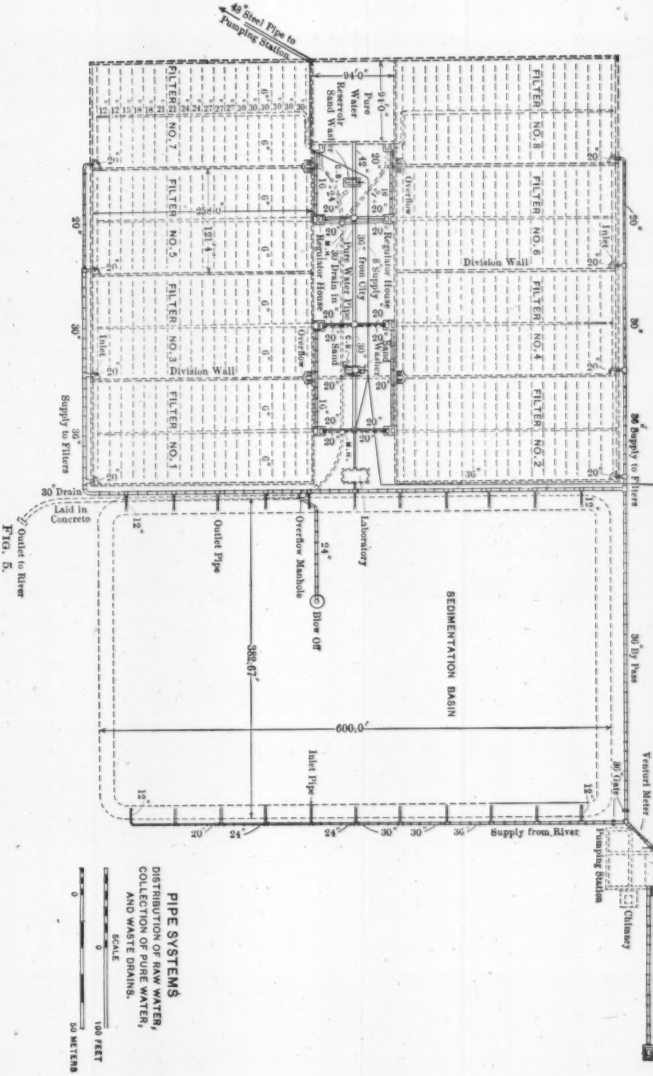

48" Steel Pipe to Pumping Station
FILTER NO. 7
FILTER NO. 5
FILTER NO. 3
FILTER NO. 1
FILTER NO. 8
FILTER NO. 6
FILTER NO. 4
FILTER NO. 2
Pure Water Reservoir
Sand Washer
Regulator House
Pure Water Pipe
30" Drain in Regulator House
Sand Washer
Overflow
Inlet
Division Wall
Supply to Filters
30" Drain
Laid in Concrete
Outlet to River
Outlet Pipe
Overflow Manhole
Laboratory
Blow Off
SEDIMENTATION BASIN
382.67'
600.0'
Inlet Pipe
Supply from River
36" Supply to Filters
36" By Pass
4" Supply from City
Venturi Meter
36" Gate
Pumping Station
Chimney
Pump Well
Intake
PIPE SYSTEMS
DISTRIBUTION OF RAW WATER, COLLECTION OF PURE WATER, AND WASTE DRAINS.
SCALE
100 FEET
50 METERS

FIG. 5.

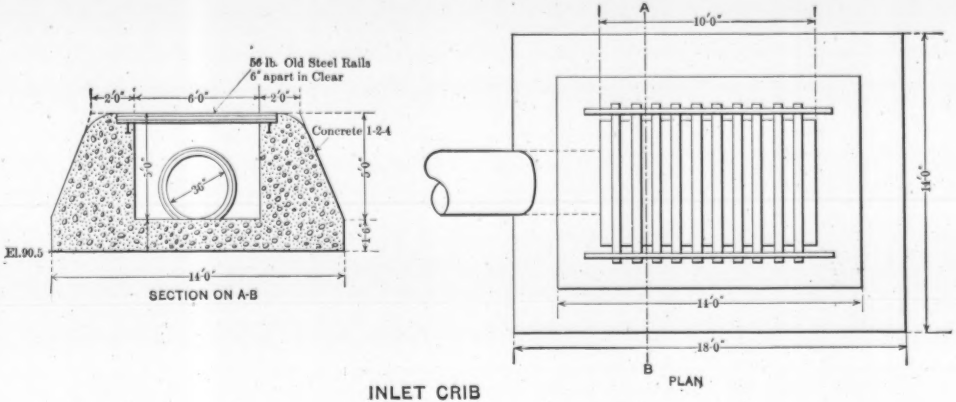
The pumping station building, to a point above the highest flood-level, is of massive concrete construction, without openings. Nearly all the machinery is necessarily below this level, and in high water the sluice gates are closed, and the machinery is thus protected from flooding. The superstructure is of pressed brick, with granite trimmings. The general form of the pumping station and the arrangement of the pumping machinery are shown in Fig. 7. A distant view of the building is shown in Fig. 1, Plate XXXIV.

Meter for Raw Water.—Upon leaving the pumping station the water passes through a 36-in. Venturi meter having a throat diameter of 17 ins., the throat area being two-ninths of the area of the pipe. The meter records the quantity of water pumped, and is also arranged to show on gauges in the pumping station the rate of pumping.

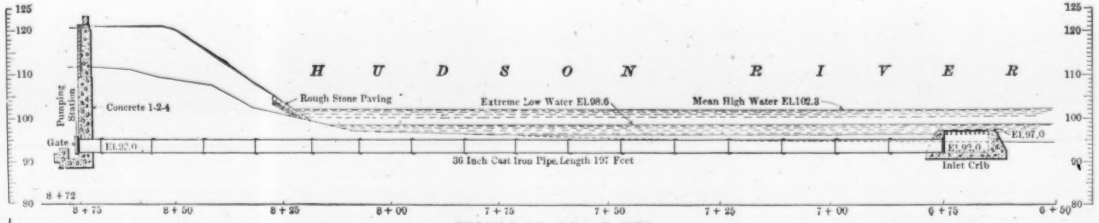
Aeration.—After leaving the meter, the water passes to the sedimentation basin through eleven outlets. These outlets consist of 12-in. pipes on end, the tops of which are 4 ft. above the nominal flow line of the sedimentation basin. Each of these outlet pipes is pierced with 296 $\frac{3}{8}$-in. holes extending from 0.5 to 3.5 ft. below the top of the pipe. These holes are computed so that when 11 000 000 galls. of water per day are pumped, all the water will pass through the holes, the water in the pipes standing flush with the tops. The water is thus thrown out in 3 256 small streams, and becomes aerated. When more than the above amount is pumped, the excess flows over the tops of the outlet pipes in thin sheets, which are broken by the jets.

Regarding the necessity for aeration, no observations have been taken upon the Hudson River, but, judging from experience with the Merrimac, at Lawrence, where the conditions are in many respects similar, the water is at all times more or less aerated, and, for the greater part of the year, it is nearly saturated with oxygen, and aeration is not necessary. During low water in summer, however, there is much less oxygen in the water, and at these times aeration is a distinct advantage. Further, the river water will often have a slight odor, and aeration will tend to remove it. The outlets are arranged so that they can be removed readily in winter, if they are not found necessary at that season.

Sedimentation Basin.—The sedimentation basin has an area of 5 acres and is 9 ft. deep. To the overflow, it has a capacity of 14 600 000 galls., and, to the flow line of the filters, 8 900 000 galls. There is thus



INLET CRIB



PROFILE OF INTAKE PIPE

FIG. 6.

a reserve capacity of 5 700 000 galls. between these limits, and this amount can be drawn upon, without inconvenience, for maintaining the filters in service while the pumps are shut down. This allows a freedom in the operation of the pumps, which would not exist with the water supplied direct to the filters.

The sedimentation basin is built on the river bank, largely above the natural surface of the soil. The sides are embankments made of clay obtained in excavating for the filters, mixed with gravel dredged from the river. These materials were put down in alternate 3-in. layers, wet, and harrowed, and were rolled with 3-ton grooved rollers on the top of each gravel layer until the gravel was forced down into and thoroughly embedded in the clay. The embankments made in this way are extremely solid, stand up in vertical sections when cut, are not readily washed, and no leakage through them has appeared at any point. The outsides of the embankments are covered with soil, and the inside and bottom with 16 ins. of puddle, which is protected from frost on the sides by covering with gravel, above which is a rough blue-stone pavement.

The puddle was made by mixing equal volumes of the clay obtained in excavating for the filters, and mixed sand and gravel obtained from the river by dredging. It differed from the material of the embankments only in more thorough mixing, and greater care in placing. The materials were mixed in a pug mill. It was soon found that the best mixing was secured with rather large quantities of water, while the best ramming required that the materials should not be too wet. Accordingly, the materials were mixed wet, given a preliminary ramming, allowed to stand two or three days, or as long as was necessary, depending upon the weather, and afterward given the final ramming. If the whole became too dry, in the interval, it was moistened at this time. The puddle was put down in three layers, and the concrete rested directly upon it. The concrete was put down in blocks about 7 ft. square, with $\frac{1}{2}$-in. joints, extending half-way from top to bottom, filled with asphalt. The maximum rate of placing puddle was about 3 000 cu. yds. per month.

The water enters the sedimentation basin from eleven inlets along one side and is withdrawn from eleven outlets directly opposite. The inlets and aerating devices described previously bring the water into the basin without current, and evenly distributed along one side.

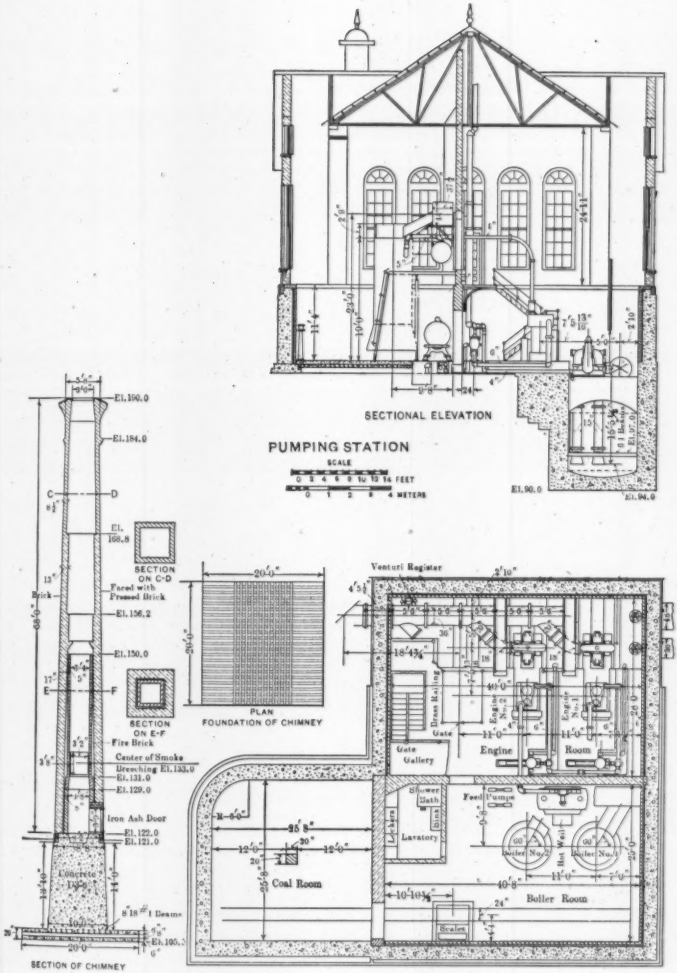

SECTIONAL ELEVATION
PUMPING STATION
SCALE
0 2 4 6 8 10 12 14 FEET
0 1 2 3 4 METERS
El. 90.0
El. 94.0
El. 190.0
El. 184.0
El. 168.8
SECTION ON C-D
Faced with Pressed Brick
El. 156.2
El. 150.0
SECTION ON E-F
Fire Brick
Center of Smoke Breeching El. 133.0
El. 131.0
El. 129.0
Iron Ash Door
El. 122.0
El. 121.0
Concrete
8 18 # I Beams
El. 105.3
SECTION OF CHIMNEY
PLAN
FOUNDATION OF CHIMNEY
Venturi Register
Drass Railing
Gate
Gate Gallery
Engine Room
Engine No. 2
Engine No. 1
Shower Bath
Sink
Lavatory
Feed Pumps
Boiler No. 2
Boiler No. 1
Hot Well
Coal Room
Boiler Room
Scales

FIG. 7.

Both inlets and outlets are controlled by gates, so that any irregularities in distribution can be avoided. The floor of the sedimentation basin is built with even slopes from the toe of each embankment to a sump, the heights of these slopes being 1 ft., whatever their lengths. The sump is connected with a 24-in. pipe leading to a large manhole in which there is a gate through which water can be drawn to empty the basin. There is an overflow from the basin to this manhole, which makes it impossible to fill the basin above the intended level. A section of the embankment about the sedimentation basin and other details are shown in Fig. 8. The method of placing the concrete floors is shown in Fig. 2, Plate XXXII, while a view of a portion of the finished basin in use is shown in Fig. 1, Plate XXXIV.

When the basin is being cleaned, the supply is maintained by opening the by-pass from the pumps to the filters and pumping to them direct. Cleaning can be done during periods of continued clear weather, such as occur in the summer and fall, without the slightest detriment to the filters, and it will not be necessary to clean it oftener than once a year.

No special provision is made for flushing out the mud; but, by opening the gates on the inlet and outlet pipes, water can be introduced at twenty-two points along two sides of the basin, and it is believed that this, with the slope of the bottom, will be sufficient to enable the mud to be swept out readily.

Filters.—The filters are of masonry, and are covered to protect them against the winters, which are quite severe in Albany. The piers, cross-walls and linings of the outside walls, entrances, etc., are of vitrified brick. All other masonry is concrete. The average depth of excavation for the filters was 4 ft., and the material at the bottom was usually blue or yellow clay. In some places shale was encountered. In one place soft clay was found, and there the foundations were made deeper.

Floors.—The floors consisted of inverted, groined, concrete arches, arranged to distribute the weight of the walls and vaulting over the whole area of the bottom. The bottoms were put in in alternate squares running diagonally with the pier lines, as shown in Fig. 1, Plate XXX. In this way forms could be used giving the shape of the arches, and the surface of the concrete was brought to the required form by screeds. After the concrete had set, the forms were removed

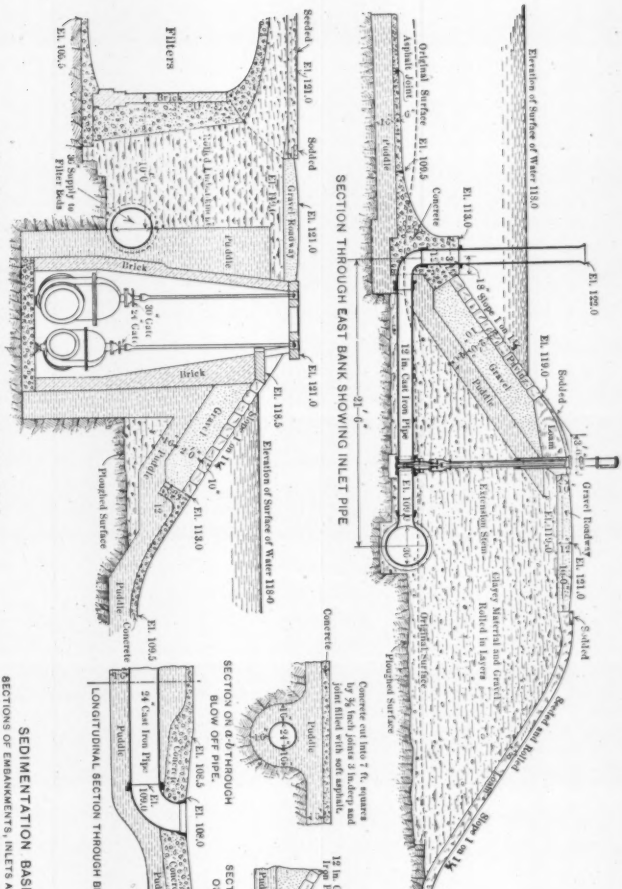

SECTION THROUGH EAST BANK SHOWING INLET PIPE
SECTION THROUGH WEST BANK AND OVERFLOW MANNHOLE
SECTION ON a-b THROUGH BLOW OFF PIPE.
SECTION THROUGH OUTLET PIPE.
LONGITUDINAL SECTION THROUGH BLOW OFF PIPE
SEDIMENTATION BASIN
SECTIONS OF EMBANKMENTS, INLETS AND OUTLETS
SCALE OF FEET
SCALE OF METERS
Elevation of Surface of Water 118.0
Original Surface
Asphalt Joint
Concrete
Puddle
Sodded
Gravel Roadway
Loam
Extension Stem
Clayey Material and Gravel Rolled in Layers
Original Surface
Ploughed Surface
Sodded and Rolled
Slope 1 on 1½
Concrete cut into 7 ft. squares by ¾ inch joints 3 in. deep and joint filled with soft asphalt.
12 in. Cast Iron Pipe
Removable Casting
24" Cast Iron Pipe
Filters
Brick
Gravel
Elevation of Surface of Water 118.0
Ploughed Surface
30" Supply to Filter Beds
30" Gate
24" Gate
Gravel Roadway
Sodded
Puddle
Brick
El. 122.0
El. 121.0
El. 119.0
El. 118.0
El. 116.0
El. 112.0
El. 109.5
El. 108.5
El. 108.0
El. 106.5
21'-6"

FIG. 8.

and the concrete for the intermediate blocks was placed, using screeds to bring the surfaces to the lines given by the blocks already in position. The foundations for the outside walls were put in in alternate sections, the lengths of the sections being usually about 10 ft.

Walls.—For the outside walls the brick linings, 8 ins. thick, were built first to the full height. A certain number of bricks were laid endways, and projected into the concrete. The projecting bricks occupied about 4% of the area of the wall. Afterward, wooden forms were put up on the outside, and the concrete backing was filled in. Sections of the walls are shown in Fig. 10. The arrangement of the projecting brick is shown in Fig. 1, Plate XXXII, which also shows the outside forms for the concrete wall in the distance.

In stopping the day's work on the outside walls, the concrete was stepped off, the horizontal joints being made at least four times as long as the vertical joints. This bonding proved adequate, and no cracks have followed the joints made in this way. The horizontal joints were strengthened further by driving a wooden stick into the concrete before stopping the work. This was removed afterward, and, when more concrete was added, it formed a thick tongue in the old work. By using sticks, in which deep grooves were cut, and with a slight batter, they could be taken out without trouble and used again indefinitely.

The outside walls are thus practically monolithic, and differ in this respect from the floors and vaulting, which are made in sections, the dimensions of which do not exceed 14 ft.

Vaulting.—The concrete vaulting was placed on wooden centers supported on wedges which could be knocked out after the concrete had set, so that the centers came down readily, and could be moved forward and used again. Some of the centers were used four or five times in the course of the work, the only repairs necessary being the patching of the lagging, and they were in good order at the end of the work. The vaulting was designed with a clear span of 12 ft., a rise of 2½ ft., and a thickness of 6 ins. at the crown, but the clear span was reduced to 11 ft. 11 ins. to fit the sizes of the bricks in the piers. It was put in in squares, the joints being on the crowns of the arches parallel with the lines of the piers, and each pier being the center of one square. The manholes are in alternate sections, and are of concrete, built in steel forms with castings at the tops, securely jointed to the concrete.

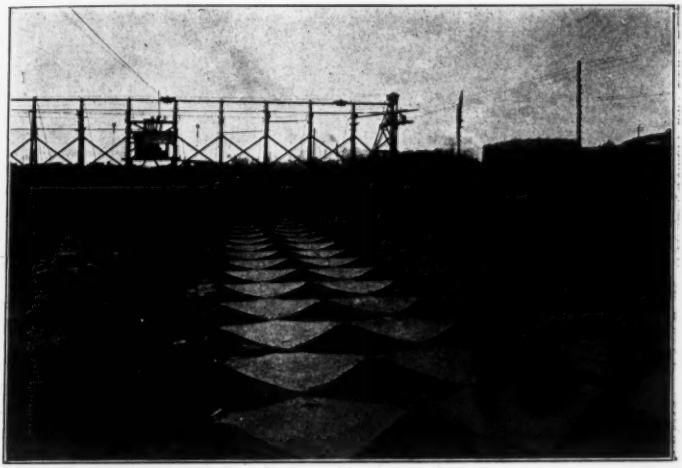


FIG. 1.—PLACING THE FLOOR OF A FILTER.

FIG. 2.—BUILDING THE BRICK PIERS.

As the centers for the vaulting were moved forward, it was necessary that they should fit closely the piers and walls in their new positions. The lumber for the centers was cut by machinery, and was of uniform dimensions. Considerable care was required to make sure that the distances between the piers, and between the piers and the walls, should always be exactly right to fit the woodwork. This was accomplished by stopping the work two courses before the top, and giving new lines and grades at this elevation, after which the two upper courses were placed. An opportunity was thus given to correct any slight variations which might have crept in below this level. This procedure was much facilitated on the walls by projecting the upper courses of brick about $\frac{1}{2}$ in., making a slight cornice. Variations of $\frac{1}{4}$ in. or so below this level do not show. Such projections were not used on the piers in this case, but their use would have added to the convenience of construction and the appearance of the finished work.

Above the vaulting there are 2 ft. of earth and soil, grassed on top. The tops of the manholes are 6 ins. above the soil to prevent rain water from entering them. The drainage of the soil is effected by a depression of the vaulting over each pier, partially filled with gravel and sand, from which water is removed by a 2-in. tile drain going down the center of the pier and discharging through its side just above the top of the sand in the filter. The saving in cost by this arrangement was considerable, as the cost of the drains was much less than that of the concrete which would have been necessary to fill the areas over the piers had any other system been adopted. Further, the water entering in this way is as good as any water available, and there is every reason for adding it to the supply. It enters the supply before it passes through the filters. Sections of the vaulting are shown in Fig. 10; details of the centering used are shown in Fig. 13. In Plate XXXI, Fig. 1 shows the placing of concrete, and Fig. 2 gives a general view of the vaulting at various stages. The finished vaulting is shown from beneath in Plate XXXIII, Figs. 1 and 2.

In order to provide ready access to each filter, a part of the vaulting near one side is elevated and made cylindrical in shape, making an inclined runway from the sand level to a door, the threshold of which is 6 ins. above the level of the overflow. This sand-run is provided with permanent timber runways and with secure doors.

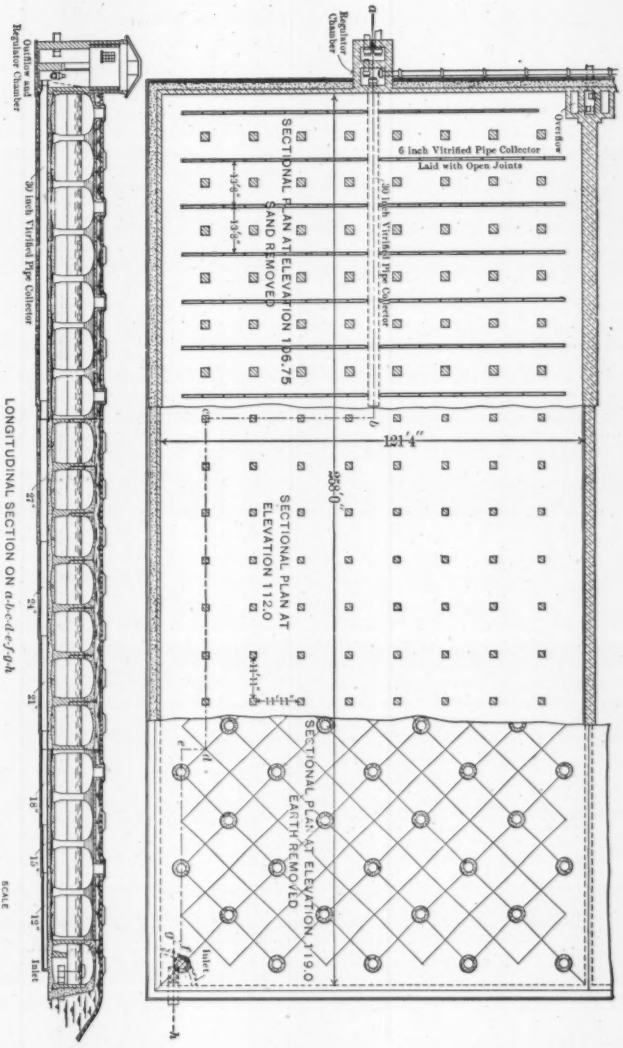
The manholes of the filters are provided with double covers of steel

plates to exclude the cold. The covers also exclude light. When cleaning the filters, light can be admitted by removing the covers. Supports for electric lights are placed in the vaulting, so that the filters can be lighted by electricity and the work of cleaning can be done at night, and in winter under heavy snow, without removing the covers. The electric lights have not yet been installed. The supports for this purpose are castings, which were put at regular places on the centers and surrounded by concrete in the vaulting. They are strong enough, and are placed so as to serve for an overhead carrier system for removing the sand, should that be found advantageous in the future. For the present, the plan is to run out the sand in wheel-barrows, as is commonly done in European filter plants, and thus far in this country. The possibility of putting overhead rails with suspended buckets for carrying out the dirty sand seemed quite promising, but prices secured on the installation of machinery were so great as not to justify it for the removal of the quantity of sand estimated as necessary to be taken out from the filters.

The regulator houses, the entrances to the sand-runs, and all exposed work are of pressed brick, with Milford granite trimmings and slate roofs. The regulator houses have double walls and double windows, and a tight ceiling in the roof, to make them as warm as possible and to avoid the necessity of artificial heat to prevent freezing.

The vaulting is similar, in many respects, to that of the covered filters at Ashland, Wis., and at Somersworth, N. H., but differs from that vaulting, in that it is entirely of concrete instead of brick backed by concrete.

Underdrains.—The main underdrains for removing the filtered water are of vitrified pipe surrounded by concrete, and are entirely below the floors of the filters. These drains were put in before the construction of the filters was commenced, and the concrete surrounding them was brought to the plane of the bottom of the foundations, so that when the floor was built it went over them continuously, without breaking in any way the line of inverted arches. This arrangement was adopted because the drains would have been in the way if they had been placed entirely above the floor, and if any part of the drain had been placed in the normal floor-span it would have reduced the strength of the inverted arches, and might itself have been broken by their pressure. As the surrounding material is clay or tight



FILTER BEDS

PLAN AND SECTION OF FILTER NO. 2

FIG. 9.

rock, no danger of loss of water by seepage can result from this arrangement.

The main effluent collectors are 30-in. vitrified pipes, reduced by castings to 20 ins. at the outlets, and the effluent from each filter passes through a 20-in. gate. The underdrains are made much larger than would ordinarily be required for carrying the quantities of water involved. The reason for this is that, after a filter has just been scraped, the friction of the water in passing the sand is very slight. If the friction of the underdrainage system is not kept very low, there will be so much loss of head that when a filter is started the pull exerted at remote parts of the filter will be less than at points near the outlet, and thus the parts near the outlet will operate at rates which are too high, while the more remote parts will hardly filter at all, and the resulting purification is less than it should be. The underdrainage system is so designed that, when starting a filter after cleaning, the friction of the sand is about 50 mm. at a rate of 3 000 000 galls. per acre daily, and the friction of the underdrainage system is estimated at 10 mm. This very low friction, which is necessary, is obtained by the use of ample sizes for the underdrains and low velocities in them. In the outlet and measuring devices moderate losses of head are not objectionable, and the sizes of the pipes and connections are, therefore, smaller than the main underdrains.

Connections with the drain are made through thirty-eight 6-in. outlets in each filter, passing through the floor and connected with 6-in. lateral drains running through the whole width of the filter. These drains were made with pipes having one side of the bell cut off, so that they would lie flat on the floor and make concentric joints, without support and without having to be wedged. They were laid with a space of about 1 in. between the barrels, leaving a large opening for the admission of water from the gravel. The general arrangement of the drainage system is shown in Fig. 5. Other details are shown in Figs. 9 and 10, while the computed frictional resistance of one filter is shown in Fig. 11.

Filter Gravel.—The gravel surrounding the underdrains is of three grades. The material was obtained from the river-bed by dredging, and was of the same stock as that used for preparing ballast for the concrete. It was separated and cleaned by a special, cylindrical, revolving screen. The coarsest grade of gravel was that which would not

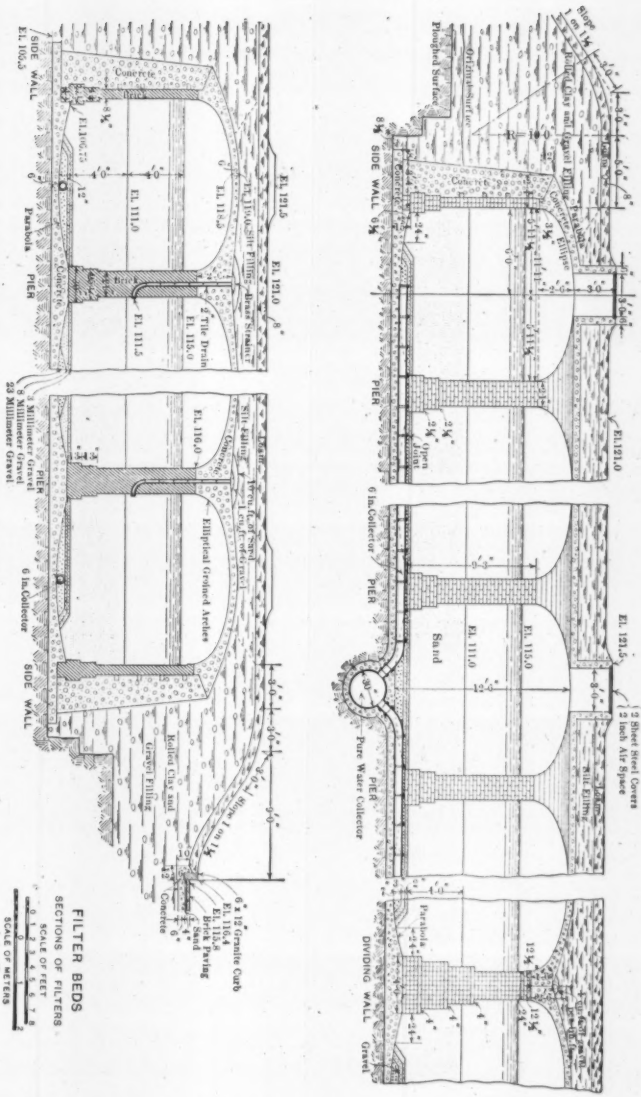

FILTER BEDS
SECTIONS OF FILTERS
SCALE OF FEET
SCALE OF METERS
SIDE WALL
PIER
DIVIDING WALL
Concrete
Parabola
Ellipse
Ploughed Surface
Orifice Surface
Rolled Clay and Gravel Filling
Silt Filling
Sand
Gravel
Pure Water Collector
6 in. Collector
2 Sheet Steel Covers
2 Inch Air Space
Open Joint
Brass Strainer
2 Tile Drain
Elliptical Groined Arches
Gravel Filling
Rolled Clay and Gravel Filling
6 x 12 Granite Curb
Brick Paving
Sand
Concrete
8 Millimeter Gravel
23 Millimeter Gravel
EL. 121.5
EL. 121.0
EL. 119.0
EL. 118.5
EL. 116.4
EL. 116.0
EL. 115.0
EL. 111.0
EL. 111.5
EL. 106.75
EL. 105.5

FIG. 10.

pass round holes 1 in. in diameter, and free from stones more than about 2 ins. in diameter. At first it was required to pass a screen with holes 2 ins. in diameter, but this screen removed many stones which it was desired to retain, and the screen was afterward changed to have holes 3 ins. in diameter. The intermediate grades of gravel passed the 1-in. holes, and were retained by a screen with round holes $\frac{3}{8}$ in. in diameter. The finest gravel passed the above screens and was retained by a screen with round holes $\frac{3}{16}$ in. in diameter. The gravel was washed, until free from sand and dirt, by water played upon it during the process of screening, and it was afterward taken over screens in the chutes where it was separated from the dirty water, and, when necessary, further quantities of water were played upon it at these points.

The average mechanical analyses of the three grades of gravel are shown by Fig. 14. Their effective sizes were 23, 8 and 3 mm., respectively, and, for convenience, they are designated by these numbers. The average uniformity coefficent for each grade was about 1.8.

The 23-mm. gravel entirely surrounded the 6-in. pipe drains, and was carried slightly above their tops. In some cases it was used to cover nearly the whole of the floor, but this was not insisted upon.

The 8-mm. gravel was obtained in larger quantity than the other sizes, and was used to fill all spaces up to a plane $2\frac{1}{2}$ ins. below the finished surface of the gravel, this layer being about 2 ins. thick over the tops of the drains, and somewhat thicker elsewhere.

The 3-mm. gravel was then applied in a layer $2\frac{1}{2}$ ins. deep, and the surface leveled. The grades for the two upper gravel layers were shown directly by the joints in the brick work of the piers.

The form of construction made it best to put a lateral drain in each section, or 13 ft. 8 ins. apart on centers. The drain itself occupies 7 ins., and the longest course which water has to pass in the gravel, in any event, is about $6\frac{1}{2}$ ft. This distance is so short that the frictional resistance of the filtered water in passing through the gravel is extremely small, and, therefore, it was possible to vary the gravel sections somewhat according to the relative amounts of gravel of the several grades obtained in screening, without detriment to the work. The thickness of the 3-mm. gravel was varied between 2 and $2\frac{1}{2}$ ins., according to the supply available, and similar variations were made in the other grades; but the finished surface of the gravel was always kept

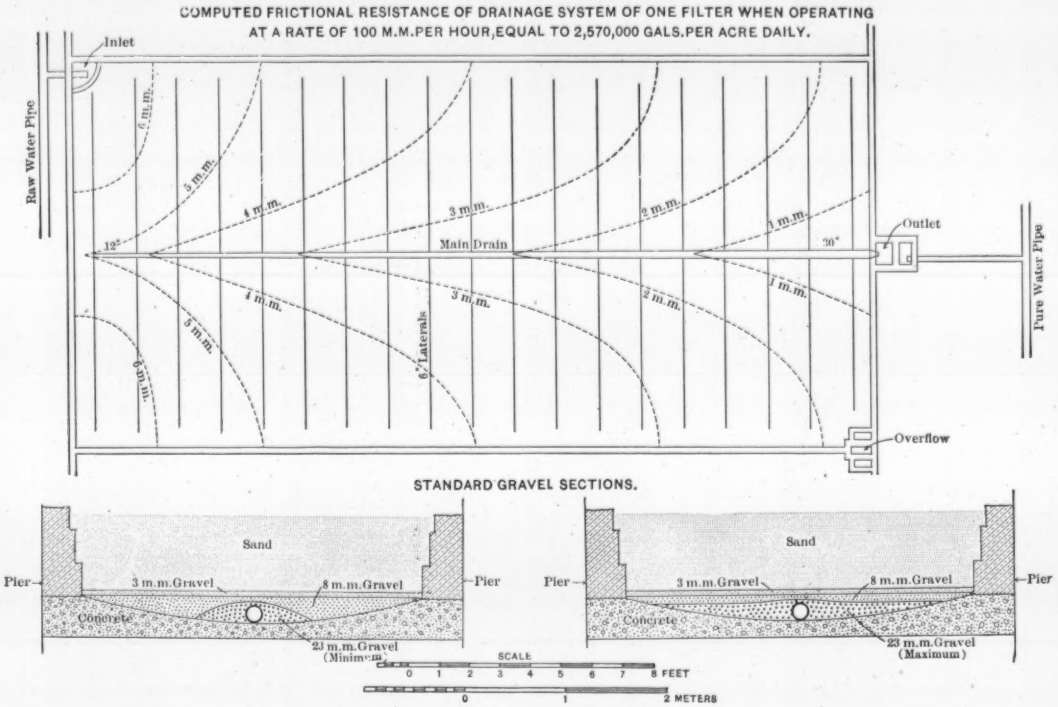

COMPUTED FRICTIONAL RESISTANCE OF DRAINAGE SYSTEM OF ONE FILTER WHEN OPERATING
AT A RATE OF 100 M.M.PER HOUR, EQUAL TO 2,570,000 GALS.PER ACRE DAILY.
Inlet
Raw Water Pipe
6 m.m.
5 m.m.
4 m.m.
3 m.m.
2 m.m.
1 m.m.
12"
Main Drain
30"
Outlet
Pure Water Pipe
6" Laterals
Overflow
STANDARD GRAVEL SECTIONS.
Sand
Pier
3 m.m.Gravel
8 m.m.Gravel
Concrete
23 m.m.Gravel
(Minimum)
23 m.m.Gravel
(Maximum)
SCALE
0 1 2 3 4 5 6 7 8 FEET
0 1 2 METERS

FIG. 11.

at the same elevation. Typical sections of the arrangements of the gravels are shown in Figs. 10 and 11, and in Fig. 1, Plate XXXIII. The greatest rate at which gravel was obtained and placed was about 1 500 cu. yds. per month.

Filter Sand.—The preliminary estimates of cost were based upon the use of filter sand from a bank near the filter site. Further examination showed that this sand contained a considerable quantity of lime, and it was found by experiment with a small filter constructed for that purpose that the use of this sand would harden the water by about 2 parts in 100 000, and the amount of lime contained in the sand, namely, about 7%, was sufficient to continue this hardening action for a considerable number of years. This was regarded as a serious objection to its use, and the specifications were drawn limiting the amount of lime in the sand. This excluded all of the local bank sands. The river sands which were used were nearly free from lime, and, in the end, the sand as secured was probably not only free from lime, but more satisfactory in other ways, and also cheaper than the bank sand would have been.

The sand was obtained from the river at various places by dredging. It was first taken up by dipper-dredges, and brought in scows to a point in the back channel a little north of the filter plant. It was there dumped in a specially prepared place in the bottom of the river, from which it was lifted by a hydraulic dredge and pumped through a 15-in. pipe an average distance of 525 ft. to points selected, and varied from time to time, on the flats north of the filters. The water containing the sand was then put through screens having meshes which excluded all stones 5 mm. in diameter and over, and was then taken into basins where the sand was deposited in conical-shaped piles, the water running over the surface and away. The boards around the edges were always kept so low as to prevent ponding and the consequent deposition of fine particles of dirt with the sand. After a certain amount of sand was deposited in one place, the point of discharge was changed, the pile drained, inspected, and, if found satisfactory, carried to the filters.

As the work progressed, an improvement upon this method was made. Piles were driven throughout the area in which the sand was deposited, a bridge was constructed, and the sand trucks passed over it. Sand was shoveled continuously into the trucks, new sand

FIG. 1.—PLACING THE CONCRETE VAULTING.

FIG. 2.—GENERAL VIEW OF VAULTING, UNDER CONSTRUCTION.

taking the place of that shoveled out. Afterward, a steam dredge was installed, which took the place of a part of the shovelers. With this arrangement the sand was delivered constantly in one place, and the necessity for changing the position of the screen was avoided.

A considerable amount of the finer part of the sand was removed by this process and accumulated below. This material was not suitable for filter sand and was wasted. Some of it was afterward used for filling. The gravel removed from the sand was used under the pavement of the sedimentation basin, and a part of it was screened for filter gravel. The capacity of the dredge was about 500 cu. yds. per day, scow measurement; but owing to the losses and wastes, and the greater compactness of the sand in the filters, the amount of sand secured, as shown by the final measurements, did not exceed 7 500 cu. yds. per month.

The specifications of the filter sand require that:

"The filter sand shall be clean river, beach or bank sand, with either sharp or rounded grains. It shall be entirely free from clay, dust or organic impurities, and shall, if necessary, be washed to remove such materials from it. The grains shall, all of them, be of hard material, which will not disintegrate, and shall be of the following diameters: Not more than 1% by weight, less than 0.13 mm, nor more than 10% less than 0.27 mm.; at least 10% by weight shall be less than 0.36 mm. and at least 70%, by weight, shall be less than 1 mm., and no particles shall be more than 5 mm. in diameter. The diameters of the sand grains will be computed as the diameters of spheres of equal volume. The sand shall not contain more than 2%, by weight, of lime and magnesia taken together and calculated as carbonates."

With the river sand and the method of handling adopted by the contractors, it was possible to control the quality of the sand, so that the specifications were complied with. In the lower layers of two of the filters a little sand was allowed which contained a few particles above 5 mm. in diameter. The screens were adjusted afterward so that the largest remaining particles were less than 4 mm. in diameter. The filter sand has effective sizes of from 0.29 to 0.32 mm., averaging 0.31; and uniformity coefficients from 2.2 to 2.5, averaging 2.3. The voids in the sand when closely packed amount to about 40% of the total volume. Its mechanical composition is shown by the diagram, Fig. 14.

The sand and also the gravel were delivered in the filters through the manholes, temporary plank roadways being built for that purpose. Trucks carrying 1½ yds., with a pair of horses, were driven over the

vaulting in all directions, without hesitation and without damage to it. The sand was dumped on plank platforms constructed below. A record was kept of all the planks used for this purpose, and they were required to be taken up in the presence of inspectors afterward to prevent the possibility of leaving any of them in the filter. This arrangement necessitated working over all the sand underneath the points of dumping, and it thereby became loosened from the excessive packing caused by dumping it from a height. The sand was deposited in three horizontal layers, so that, if by accident sand of unusual quality was placed at any point, the same kind of sand would not extend from top to bottom. The method of placing the sand in layers is shown in Fig. 1, Plate XXXIII, which also shows the gravel layers and a lateral underdrain; while a completed filter with the sand smoothed ready for use is shown in Fig, 2, Plate XXXIII.

Sand-Washing Apparatus.—Most of the suspended matters in the filtered water are held by the top layer of sand, and this layer is removed from time to time. The dirty sand is washed, and eventually replaced in the filters. Two ejector sand-washing machines, shown in Fig. 15, are provided at convenient places between the filters. In them the dirty sand is mixed with water, and is thrown up by an ejector, after which it runs through a chute into a receptacle, from which it is again lifted by another ejector. It passes in all through five ejectors, part of the dirty water being wasted each time. The sand is finally collected from the last ejector, where it is allowed to deposit from the water.

Sand washers of this kind have been used for many years by some of the London water companies, and more recently at Hamburg; and also at Lawrence and Poughkeepsie in this country. The entire central court between the filters and about the sand washers, shown in Fig. 2, Plate XXXIV, is paved with brick upon a concrete foundation, and affords a convenient space for handling and storing sand.

Inlets to Filters.—Water is admitted to each filter through a 20-in. pipe from a pipe system connecting with the sedimentation basin. Just inside of the filter wall is placed a standard gate and beyond that a balanced valve connected with an adjustable float to shut off the water when it reaches the desired height on the filter (Fig. 12). These valves and floats were constructed from special designs, and are similar in principle to valves used for the same purpose in the Berlin water filters.

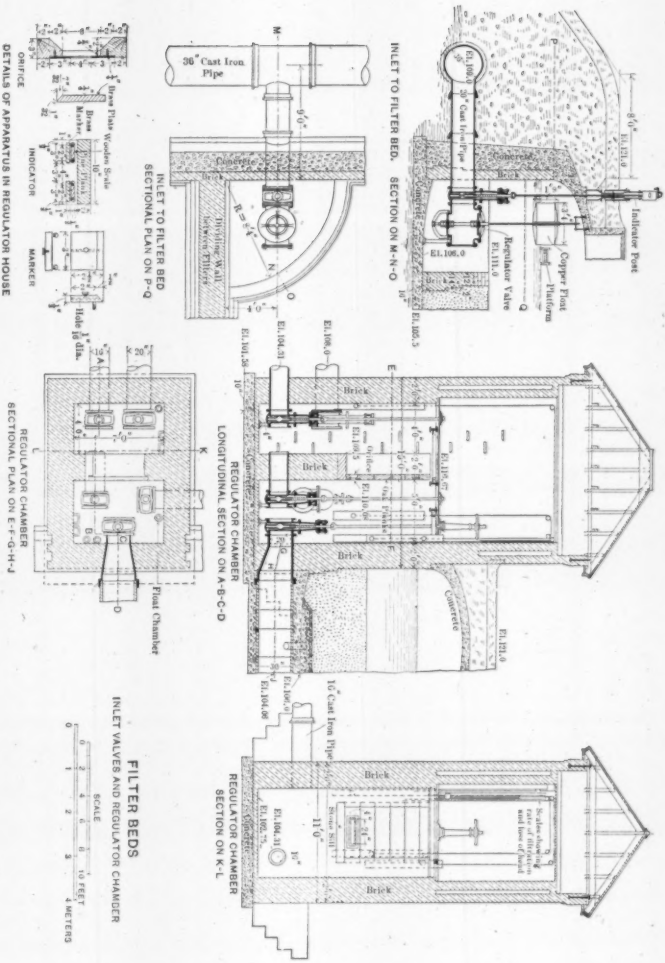

INLET TO FILTER BED. SECTION ON M-N-O
INLET TO FILTER BED SECTIONAL PLAN ON P-Q
DETAILS OF APPARATUS IN REGULATOR HOUSE
REGULATOR CHAMBER LONGITUDINAL SECTION ON A-B-C-D
REGULATOR CHAMBER SECTIONAL PLAN ON E-F-G-H-J
REGULATOR CHAMBER SECTION ON K-L
FILTER BEDS
INLET VALVES AND REGULATOR CHAMBER
SCALE
10 FEET
4 METERS
Indicator Post
Copper Float
Platform
Regulator Valve
Concrete
Brick
Dividing Wall between Filters
36" Cast Iron Pipe
16" Cast Iron Pipe
Stone Sill
Scales showing rate of filtration and loss of head
Float Chamber
ORIFICE
INDICATOR
MARKER
Brass Plate
Wooden Scale
Brass Marker
Pine Plank
Hole 1/16" dia.

FIG. 12.

Overflows.—Each filter is provided with an overflow, so arranged that it cannot be closed, which prevents the water level from exceeding a fixed limit in case the balanced valve fails to act. An outlet is also provided near the sand run, so that unfiltered water can be removed quickly from the surface of the filter, should it be necessary, to facilitate cleaning.

Filter Outlets.—The outlet of each filter is through a 20-in. gate controlled by a standard graduated to show the exact distance the gate is open. The water rises in a chamber and flows through an orifice in a brass plate 4 by 24 ins., the center of which is 1 ft. below the level of the sand line. At the nominal rate of filtration, 3 000 000 galls. per acre daily, 1 ft. of head is required to force the water through the orifice. With other rates the head increases or decreases approximately as the square of the rate and forms a measure of it. With water standing in the lower chamber, so that the orifice is submerged, it is assumed that the same rates will be obtained with a given difference in level between the water on the two sides of the orifice, as from an equal head above the center of the orifice when discharging into air. The general arrangement of the gate-houses, including the gates for wasting the effluent to the river, for filling filters with filtered water from below, etc., is shown in Fig. 12, while outside views of gate-houses are shown in Fig. 2, Plate XXX, and Fig. 2, Plate XXXIV.

Measurement of Effluent.—In order to show the rate of filtration two floats are connected with the water on the two sides of the orifice. These floats are counterbalanced; one carries a graduated scale and the other a marker which moves in front of the scale and shows the rate of filtration corresponding to the difference in level of the water on the two sides. When the water in the lower chamber falls below the center of the orifice, the water in the float chamber is, nevertheless, maintained at this level. This is accomplished by making the lower part of the tube water-tight, with openings just at the desired level, so that when the water falls below this point in the outer chamber it does not fall in the float chamber.

To prevent the loss of water in the float chamber by evaporation, or from other causes, a lead pipe is brought from the other chamber and supplies a driblet of water to it constantly; this overflows through the openings, and maintains the water level at precisely the desired point. The floats thus indicate the difference in water level on the two sides

FIG 1.—OUTSIDE WALL, READY FOR CONCRETE BACKING.

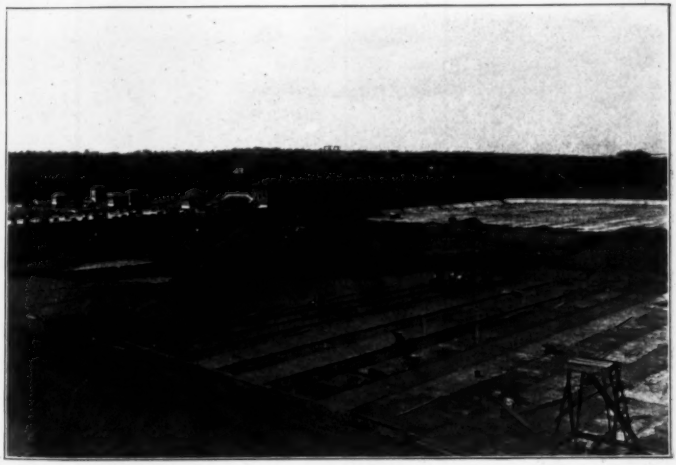


FIG. 2.—SEDIMENTATION BASIN; SHOWING CONSTRUCTION OF FLOOR.

of the orifice whenever the water in the lower chamber is above the center of the orifice; otherwise, they indicate the height of water in the upper chamber above the center of the orifice, regardless of the water level in the lower chamber. The scale is graduated to show the rates of filtration in millions of gallons per acre of filtering area. In computing this scale the area of the filters is taken as 0.7 acre and the coefficient of discharge as 0.61.

At the ordinary rates of filtration the errors introduced by the different conditions under which the orifice operates will rarely amount to as much as 100 000 galls. per acre daily, or one-thirtieth of the ordinary rate of filtration. Usually they are much less than this. The apparatus thus shows directly, and with substantial accuracy, the rate of filtration under all conditions.

Measurement of Loss of Head.—Two other floats with similar connections show the difference in level between the water standing on the filter and the water in the main drain pipe back of the gate, or in other words, the frictional resistance of the filter, including the drains. This is commonly called the loss of head, and increases from 0.2 ft. or less, with a perfectly clean filter, to 4 ft. with the filter ready for cleaning. When the loss of head exceeds 4 ft. the rate of filtration cannot be maintained at 3 000 000 galls. per acre daily with the outlet devices provided, and, in order to maintain the rate, the filter must be cleaned.

Adjustment of Gauges.—The adjustment of the gauges showing the rate of filtration and loss of head is extremely simple. When a filter is put in service the gates from the lower chamber to the pure-water reservoir and to the drain are closed, the outlet of the filter opened, and both chambers allowed to fill to the level of the water on the filter. The length of the wire carrying the gauge is then adjusted, so that the gauge will make the desired run without hitting at either end, and then the marker is adjusted. As both the rate of filtration and loss of head are zero under these conditions, it is only necessary to set the markers to read zero on the gauges to adjust them. The gates can then be opened for regular operation, and the readings on the gauges will be correct. It is necessary to use wires which are light, flexible, and which will not stretch. At first, piano wire, 0.4 mm. in diameter, was used, and was well adapted to the purpose, except that it rusted rapidly. Because of the rusting it was found

necessary to substitute another wire, and cold-drawn copper wire, 0.6 mm. in diameter, was used with fair results. Stretching is less serious than it would otherwise be, as the correctness of the adjustment can be observed and corrected readily every time a filter is out of service.

From the lower chambers in the regulator houses the water flows through gates to the pipe system leading to the pure-water reservoir. Drain pipes are also provided which allow the water to be entirely drawn out of each filter, should that be necessary for any reason, and without interfering with the other filters, or with the pure-water reservoir.

The outlets of the filters are connected in pairs, so that filtered water can be used for filling the underdrains and sand of the filters from below prior to starting, thus avoiding the disturbance which results from bringing dirty water upon the sand of a filter not filled with water.

Laboratory Building. –The scientific control of filters is regarded as one of the essentials to the best results, and to provide for this there is a laboratory building at one end of the central court between the filters and close to the sedimentation basin, supplied with the necessary equipment for full bacterial examinations, and also with facilities for observing the colors and turbidities of raw and filtered waters, and for making such chemical examinations as may be necessary. This building also provides a comfortable office, darkroom and storage room for tools, etc., used in the work.

Pure-Water Reservoir.—A small pure-water reservoir, 94 ft. square, and holding about 600 000 galls., is provided at the filter plant. The construction is similar to that of the filters, but the shapes of the piers and vaulting were changed slightly, as there was no necessity for the ledges about the bottoms of the piers and walls; while provision is made for taking the rain water, falling upon the vaulting above, to the nearest filters instead of allowing it to enter the reservoir. The floor and roof of the reservoir are at the same levels as those of the filters.

Pure-Water Conduit.—The filtered water is taken from the pure-water reservoir to the present pumping station through 7 913 ft. of 48-in. pipe. For 5 450 ft. the pipe is laid under the Erie Canal, and for 1 837 ft. through Montgomery Street, with an average cut of 22 ft The pipe is not under pressure, and is made of mild open-hearth steel plates, $\frac{5}{16}$ in. thick, rolled by the Carnegie Steel Company, at Home-

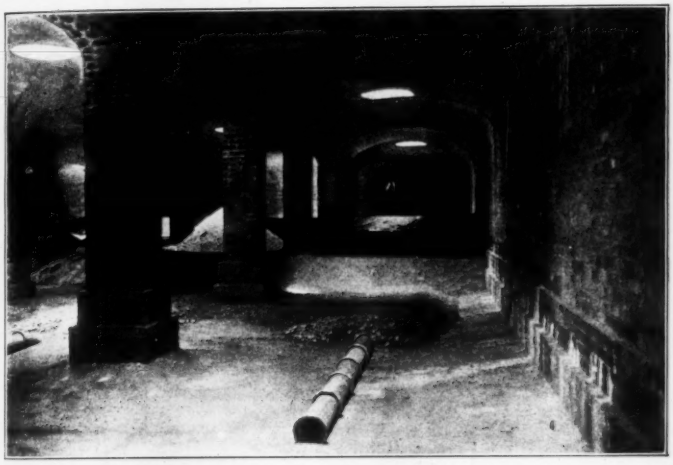


FIG. 1.—INTERIOR OF A FILTER: DRAIN, GRAVEL AND SAND LAYERS.

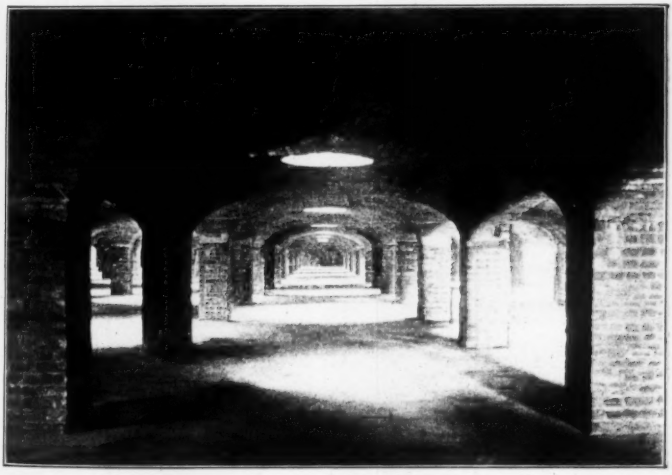


FIG. 2.—INTERIOR OF A FILTER, READY FOR USE.

stead, Pa. The pipe was made by the Carroll-Porter Boiler and Tank Company, of Pittsburg, and both plates and pipes were inspected by Stowell & Cunningham, of Albany.

Concrete Backing.—Pipe of this thickness is not stiff enough to withstand the earth pressures in the deep cut in Montgomery Street, without being badly deformed, and is not heavy enough to be safe from the danger of floating in the canal, should the water be removed from the pipe for any reason. Concrete was used to support the sides of the pipe in the deep cut and to weight it under the canal. The amount of concrete required for these purposes was considerable, and the Board decided to use a further amount in order to surround the pipe entirely. This makes a concrete pipe with a minimum thickness of 6 ins. outside of the steel pipe for practically the whole distance, capable of supporting the earth, and able itself to serve as a conduit even if the steel pipe should be removed entirely, although in that case it would not be thoroughly water-tight. For 567 ft. on hard clay bottom, the concrete on the bottom was omitted, the natural material being probably equivalent to the concrete. For about 300 ft. at the upper end, in shallow cut and where the pipe is readily accessible, the concrete was also omitted.

The hydraulic gradient of the pipe is 1 in 1 000, and the actual slope is 1 in 1 000 from the pumping station to a point where the depth is limited by the bed of the canal, the top of the pipe being always at least 4 ft. below the nominal bottom of the canal, as required by the canal authorities. It is then level until near the upper end, outside of the canal, where it rises rapidly to the pure-water reservoir.

In passing under the Patroons Creek sewer the canal authorities required that the line should leave the canal and pass under the sewer at one side, returning to the canal afterward, the object being to prevent draining the canal in case of accident to the sewer as a result of the work under it. The section out of the canal was 230 ft. long, and that part of it under the sewer was done in tunnel and below tide level, and was accomplished without accident or injury to the sewer. The entire trench under and near the sewer was back-filled with concrete.

Tightness of Work.—The specifications provided for testing each joint of the pipe under pressure as it was laid. Difficulties were found in carrying this out, and in order to prevent delay to the work, which

would have resulted in serious embarrassment, owing to the necessity of filling the canal on May 1st, this test was suspended on part of the line. In place thereof, the pipe was maintained empty after the canal was filled, and all joints were caulked on the inside until perfectly water-tight, and there was no measurable leakage from the canal into the pipe.

Coating.—The pipe was coated by dipping in asphalt, and, after caulking tight on the inside, all imperfections in the coating, due to caulking or otherwise, were covered with melted asphalt in connection with a naphtha lamp. Where the spots were not too large the asphalt on the sides came together when softened by the flame and covered the spot completely without the addition of new material. Where the uncovered patches were large this was not possible. The place was first heated, then melted asphalt was applied and thoroughly heated and melted until it incorporated itself with the old asphalt on the edges.

The greatest difficulty was experienced in repairing breaks in the coating directly on the bottom of the pipe; although these were not serious, owing to the fact that all men who worked therein were required to wear rubbers, and thus the damage done to the coating was slight. Repairs were made by building dams of cotton waste on each side of the defective place and sponging out the water, after which the plates could be heated by a naphtha lamp and repaired in the usual way.

Air Vents.—There are two considerable depressions on the line of the pipe, one under the Bridge Street drain and one under the Patroons Creek sewer. To prevent the air from obstructing the flow of water, the summits on either side of the Bridge Street drain were connected by a 3-in. pipe with expansion joints, going over the top of the sewer and allowing the air to pass by the depression without obstruction even when the pipe is nearly filled with water.

At the Patroons Creek sewer 3-in. pipes are connected with the summits on each side, going up in the manholes to a point above the hydraulic gradient of the water, and are simply left open to the air. To prevent the entrance of dirt, etc., a half-turn neck is screwed on the top, and in the outlet, facing downward, a bushing is put in, reducing the size of the hole, and making it impossible to put in any object which would obstruct the pipe.

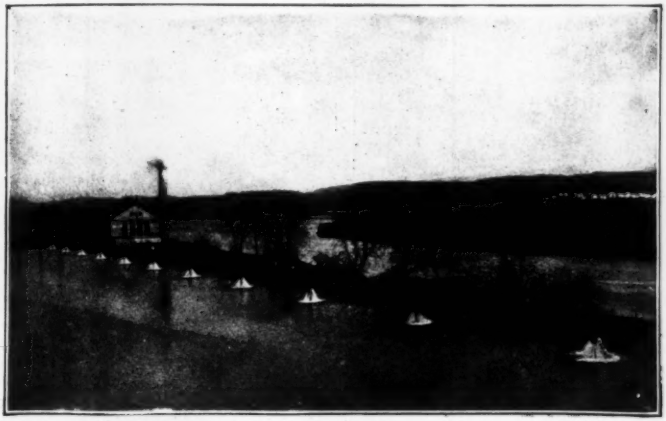


FIG. 1.—SEDIMENTATION BASIN, PUMPING STATION AND OUTLETS.

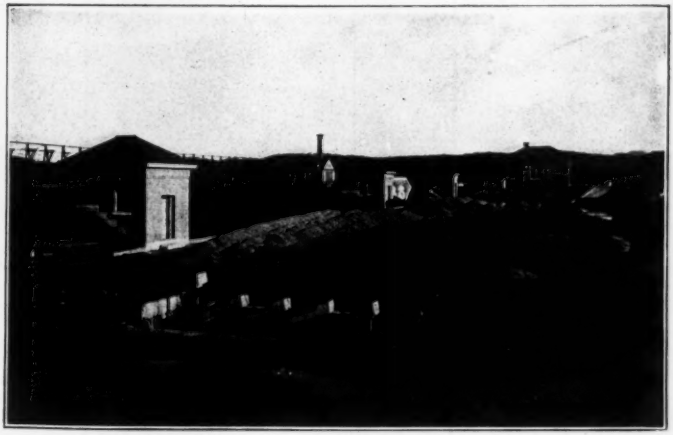


FIG 2.—CENTRAL COURT, SHOWING SAND-WASHER, DIRTY SAND, ETC.

Connections with Quackenbush Street Pumping Station.—At the lower end, connection is made with the pumping station through a rectangular brick chamber, where the flow of water can be controlled by a 48-in. gate. From this chamber the water is admitted by sluice gates to each of the three wells supplying the Allis pumps, and also to the old pump well. The entrance of water from the river was cut off by digging up the old tunnel, and building a manhole on it, in one side of which was inserted a 36-in. flange pipe and a standard 36-in. gate. The arrangement is such as to cut off the entrance of river water effectively. The gate can be opened at any time, however, if required by any emergency. The gate is accessible, and can be examined for tightness at any time by shutting off the water from the filters and lowering the water in the pump wells. Leakage is further prevented by the fact that the water in the pump wells is normally higher than the water in the river, and the tendency of leakage is outward and not inward.

Cement.—In the construction of the filters and sedimentation basin 30 000 bbls. of Atlas Portland cement were used. Iron Clad Portland cement was used in some parts of this work, and also exclusively on the pumping station and on the pure-water conduit, 8 000 or 9 000 bbls. of it being used in all. Several hundred barrels of other brands of cement were used at times when neither Atlas nor Iron Clad could be obtained promptly. No Rosendale cement was used on any part of the filters, nor in the lower part of the pumping station. On the superstructure of the regulator houses, pump house and laboratory, Rosendale cement and lime were used.

Building Sand.—All the building sand was obtained from the river by dredging, and was screened and washed by machinery. It was of the same general character as the filter sand, but had a higher uniformity coefficient and was not as clean.

Ballast for Concrete.—Gravel, obtained from the river by screening and washing, was used for the most part, but broken stone was used on various parts of the work. Sometimes the materials were used separately, but often they were mixed. The mixture was most satisfactory in every way, making a stronger and more economical concrete than either gravel or broken stone by itself.

Concrete.—The concrete for the filters was mixed by machinery in cubical boxes, 5 ft. cube, and the amount mixed was about 1.6 cu. yds.

at a time. The proportions of mixing were not fixed by the contract, but were left to the discretion of the engineer as the work progressed, and the cement was paid for separately. The usual proportions of mixing were as follows: One barrel of Portland cement, weighing 380 lbs., and nominally assumed to occupy 3.8 cu. ft.; three times this volume of sand, weighing on an average about 90 lbs. per cubic foot; and five times this volume of gravel or ballast, weighing about 100 lbs. per cubic foot, and having an average of 40% of voids. On an average for the whole work 1.26 barrels of cement were used for each cubic yard of concrete, as allowed to the contractors. Actually, the volume of concrete exceeded slightly the nominal dimensions in many cases, and the cement, reckoned on all the concrete, would be slightly less than the above figure. The discrepancies, however, were not large. The greatest rate of placing concrete on the filters was about 3 700 cu. yds. per month, and, substantially, this rate was maintained for three months.

In placing concrete around, and especially under, the 48-in. steel pipe, difficulty was found in placing that which was made with broken stone. Concrete made from gravel proved much more suitable for this purpose, and a bent rammer facilitated getting it under the pipe. Before the bent rammer was introduced, some difficulty was experienced in getting the contract sections. Afterward they were gotten easily and without trouble.

Brick Work.—All the brick work, except that in the superstructures, was of vitrified paving brick. The brick was not specified, but the specifications required brick absorbing not more than 12% of water by weight. The local brick-makers were unwilling to take the trouble to make brick hard enough to meet this requirement, and the contractors decided finally to use a second-quality paving brick. This was larger than ordinary brick, averaging 8.37 x 3.62 x 2.75 ins., and it was necessary to change the size of the piers in the filters from 20 to 21 ins., the spans of the vaulting and floor arches being reduced to correspond. These changes the contractors were allowed to make, and, in consideration of the superior quality of the brick, they were allowed an amount corresponding to the increased volume of the masonry thereby involved, although this masonry was not believed to be necessary for the stability of the structures. The bricks were regular and uniform in shape, and their use resulted in very satisfac-

tory work. The absorption ranged from 1 to 11%, and averaged 4% by weight.

The mortar used was uniformly 1 part of Portland cement to 3 parts of sand, and in a few cases, where cracks appeared and the masonry was broken, the breaks were through the bricks and not through the joints, although the bricks were extremely hard and strong. The cement was paid for separately, and about 0.80 barrel per cubic yard was used, no deduction being made for ordinary loss and waste. The greatest quantity of brick work laid in one month was 1 100 cu. yds.

Weight on Foundations.—The weight of the finished concrete was about 150 lbs. per cubic foot, as determined by tests. The brick work was computed to weigh 135 lbs. per cubic foot. The weight of the soil filling over the vaulting is estimated at 100 lbs. per cubic foot. Each of the piers in the filters, by itself, weighs 4 300 lbs. The total weight carried by the piers is about 11 tons per square foot, with the soil nearly dry. When the soil is wet, or covered with snow and ice, the weight is increased somewhat.

On the assumption that the inverted concrete arches distribute the weight of the piers over the whole area of the bottom, the weight on the foundations, of all the masonry and the earth cover, but not including the filtering material, amounts to 444 lbs. per square foot, and with the filtering material in place and with water to the nominal flow line, this weight is increased to 1 304 lbs. per square foot.

Assuming the weight of the cross-walls to be distributed on the whole width of the monolithic concrete blocks under them, the weight of the masonry, with the earth above, but without the filter sand or water, amounts to 755 lbs. per square foot, and with the sand in position and water to the flow line, to 1 415 lbs. per square foot. The excess weight under the cross-walls thus amounts to 311 lbs. per square foot with the filter empty, and to 111 lbs. per square foot with everything in position.

The foundation was generally on stiff clay capable of carrying much larger loads than the actual ones, but for a small part of the way shale was encountered. It is doubtful if there is any arch action in the floor, in which case the bulk of the weight of the piers is carried on a small area under and near the base, but the arch is there, so that if there is any tendency to settlement at any point it will come into action and

carry the weight. At one place the underlying clay was soft. Apparently, it was the channel of a stream, the position of which had been changed by the Erie Canal, and which was now filled with softer material. The area was so small as not to make it worth while to lower the general elevation of the structures, and extra foundations were provided. These were made by putting concrete, mixed in the proportion of 1: 5: 10, in the form of a wall, from hard bottom up to the ordinary foundation under the walls, and blocks of the same material, 5 ft. square, from the hard bottom up under each pier. These special foundations were put in first, the rest of the material graded to the normal level, and then the floors put in as usual. In these cases the weights on solid foundations are rather more than a ton to the square foot.

The strains in the concrete vaulting, considered as arches, are not heavy, but are somewhat difficult of computation. It appeared probable, after the centers were struck, that the vaulting did not act as arches but as a series of cantilevers. It was put in in squares, each square having a pier for a center, and the blocks were apparently strong enough to carry the loads upon them without coming to an arch action at all. Computed as cantilevers, a tensile strain of 150 lbs. per square inch is sufficient to maintain the loads as they exist, and the strength of the concrete is certainly beyond this figure.

With cold weather, some of the joints in the concrete vaulting opened slightly, indicating the absence of arch action, but no cracks in the blocks have been observed at any point.

Cracks in Walls.—The outside walls of the filters are monolithic concrete, with an 8-in. brick lining. The cross-walls are of brick. Neither has expansion joints of any kind. With cold weather, in December, 1898, cracks were discovered in each of the cross-walls on the north half of the filter plant which was then completed. One crack was found in each wall, and in each case approximately in the middle of the wall. During the winter these cracks opened from $\frac{1}{8}$ to $\frac{3}{8}$ in.

There are also a certain number of cracks on the outside walls, all of which are shown in Fig. 13. There is a break in the continuity of the outside walls where they join the regulator houses. The regulator houses were built first, and the concrete was placed against the brick work afterward, the surface of the brick being left irregular. The brick lining was bonded in, but its 8-in. thickness was not sufficient to

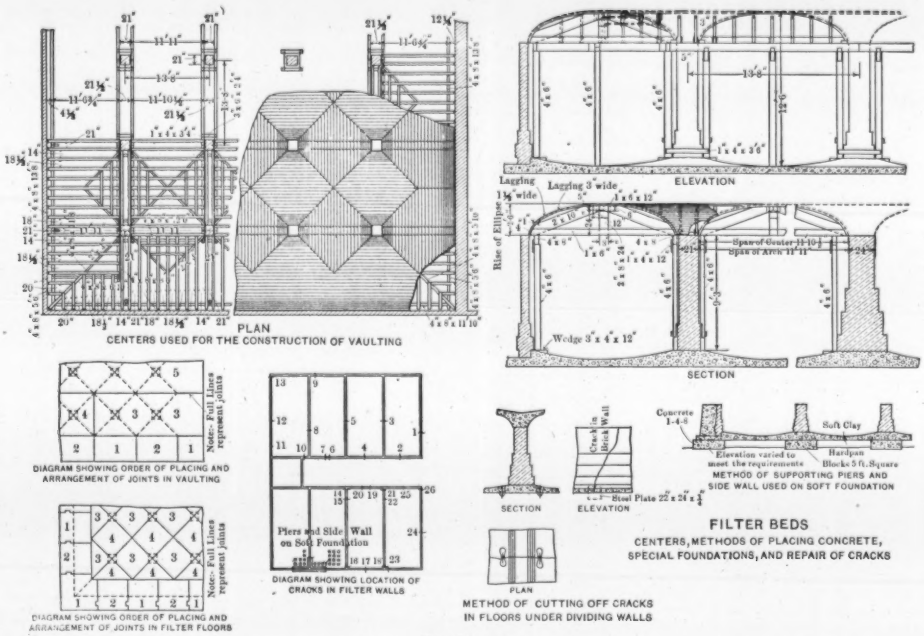

PLAN
CENTERS USED FOR THE CONSTRUCTION OF VAULTING
ELEVATION
SECTION
Lagging 1½" wide
Lagging 3" wide
Rise of Ellipse
Span of Center 11-10½
Span of Arch 11-11
Wedge 3" x 4" x 12"
DIAGRAM SHOWING ORDER OF PLACING AND ARRANGEMENT OF JOINTS IN VAULTING
Note:- Full Lines represent joints
DIAGRAM SHOWING ORDER OF PLACING AND ARRANGEMENT OF JOINTS IN FILTER FLOORS
Piers and Side Wall on Soil Foundation
DIAGRAM SHOWING LOCATION OF CRACKS IN FILTER WALLS
SECTION
ELEVATION
Steel Plate 22" x 24" x ¾"
PLAN
METHOD OF CUTTING OFF CRACKS IN FLOORS UNDER DIVIDING WALLS
Concrete 1-4-8
Soft Clay
Hardpan
Elevation varied to meet the requirements
Blocks 5 ft. Square
METHOD OF SUPPORTING PIERS AND SIDE WALL USED ON SOFT FOUNDATION
FILTER BEDS
CENTERS, METHODS OF PLACING CONCRETE, SPECIAL FOUNDATIONS, AND REPAIR OF CRACKS

FIG. 13.

withstand any heavy strain. The walls pulled away from the regulator houses at these points in eight cases, breaking the brick work, although there was no evidence of settlement. The cracks are absolutely without significance, as the clay filling on the outside is quite water-tight. The cracks in the outside walls are also unimportant, for the same reason. Where the outside walls cracked, the crack went approximately in a straight line from top to bottom through both brick and concrete, without following any joints in the concrete wall.

The cracks in the cross-walls were of more importance, for it is possible that unfiltered water might enter them above the sand and follow down through them and enter the gravel and underdrains below without filtration. This action might all take place on one side, or the water might enter the wall above on one side and leave it below on the other. This was rendered more likely, as the floors cracked at the same places and drew apart slightly, and it was actually found that, when the water was on one side of the wall only, it passed through the crack and up through the floor on the other side, some distance away from the wall. To cut this off, a hole was cut in the concrete near the wall, rather more than 2 ft. long and 8 ins. wide, with the ends widened as shown in Fig. 13. The concrete at this point was about 1 ft. thick. A steel plate $\frac{1}{4}$ in. thick and 2 ft. square was placed in the middle of this hole, and driven down until its top was flush with the top of the concrete. It thus projected about 1 ft. into the clay below, and the material was then so hard that the plate had to be severely pounded to get it into position. The hole was then filled with concrete. The steel plate is to serve as an expansion joint. If the crack opens further in the future, the concrete block will break again, approximately at the same place as the original crack, and will slip on the steel plate, the plate remaining across the crack to shut off any possible communication. Any water coming through the wall must thus come to the surface of the floor before reaching the plate, and, to prevent it from getting into the underdrains, the gravel is kept back for 5 ft. at this point, and the sand comes directly to the floor. The water on leaving the crack must then pass through at least 5 ft. of sand before it reaches the underdrains. This will insure, first, a tolerably good purification of such water; and second, if the water is dirty, a gradual clogging up and diminishing of the carrying power of the sand at this point and the final stopping of the leak. In any

event, it will be impossible for an impure water to reach the underdrains. The upper parts of the cracks, which were comparatively unimportant, and were accessible at all times, were caulked with oakum to prevent the passage of water above the sand.

These cracks appear to be clean temperature cracks, with hardly any evidence of settlement. The strength of the wall is such that, with one end free, with the temperature changes, it would pull or shove the wall over the clay without causing other cracks. The only cause of further cracks would be settlement, and this is not regarded as probable. To provide against it, however, as a possible contingency, and also to prevent the possibility of water passing down between the walls and the sand, the gravel is kept 2 ft. away from the walls all the way around, so that in case cracks occur at any points and water passes through them, it will have to pass through 2 ft. of sand. With one crack in each wall, which is virtually an expansion joint, it is not regarded as probable that other cracks will occur which will result in any openings in the floor.

Six cracks were caused by the frost in January and February, 1899. The south part of the filters was left in an uncompleted condition, and, with severe weather, the frost raised the walls in some places as much as 0.2 ft. With warmer weather the walls went back to their proper levels, but the cracks did not close entirely, and some of the brickwork was taken down, while other cracks were caulked with cement.

Without counting the slight openings at the corners of the regulator chambers, where the concrete met the brickwork, nor the six cracks caused by lifting from frost, there were twelve cracks in the whole of the work, and, as the total length of wall was 4 753 ft., there was, on an average, about one crack to each 400 ft. of wall.

Contractors' Plant.—The contractors for the filters and sedimentation basin installed a very elaborate overhead cable-carrier system for the transportation of materials to all parts of the work. This consisted of two trestles 730 ft. apart and over 900 ft. long, between which stretched four cables, each capable of carrying a load of 3 tons, at a height above all the structures on the ground, except the office and regulator houses. These cables were attached to carriages which ran on I-beams on the tops of the trestles, and power was conveyed to both ends by a system of rope drives, so that the cables could be

moved in one direction while the travelers upon the cables were moved in another direction at right angles to it. The trestles for the contractors' plant are shown in Figs. 1 and 2, Plate XXX, and in Fig. 1, Plate XXXI, and also in some of the other views. Fig. 1, Plate XXXI, also shows a skip for carrying concrete, with the carriage upon a cable. The installation of this apparatus required much longer than was anticipated, and delayed the completion of the work very much.

The system was extremely convenient, but its carrying capacity was less than was anticipated, particularly when motion in both directions was required. It was found better after a while to use the carriers, as a rule, in one direction only, and a double-track railroad was built near one of the trestles, which passed the concrete mixer and the gravel screens. Skips were moved on this railroad on trucks, received the concrete and gravel and ran to points opposite the places where the materials were to be used. The skips were then lifted from the trucks by the carrier system and carried to the required places. In this way the lateral motion of the carriers was used only occasionally, as the point at which work was being done changed. The work was so arranged that concrete or gravel was delivered in substantially the same line at one time.

Capacity of Plant and Means of Regulation.

The various filters have effective filtering areas of from 0.702 to 0.704 acre, depending upon slight differences in the thickness of the walls in different places. For the purpose of computation, the area of each filter is taken at 0.7 acre. The nominal rate of filtration is taken as 3 000 000 galls. per acre daily, at which rate each filter will yield 2 100 000 galls. daily, and, with one filter out of use for the purpose of being cleaned, seven filters normally in use will yield 14 700 000 galls. The entrances and outlets are all made of sufficient size, so that rates 50% greater than the foregoing are possible. The capacities of the intake, pumping station and piping are such as to supply any quantity of water which the filters can take, up to an extreme maximum of 25 000 000 galls. in 24 hours. The pure-water conduit from the filters to Quackenbush Street is nominally rated at 25 000 000 galls. per 24 hours, after it has become old and somewhat tuberculated. In its present excellent condition it will carry a larger quantity.

At the pumping station at Quackenbush Street there are three Allis

pumps, each capable of pumping 5 000 000 galls. per 24 hours. In addition to the above there are the old reserve pumps with a nominal capacity of 10 000 000 galls. per 24 hours, which can be used if necessary, but which require so much coal that they are seldom used. For practical purposes the 15 000 000 galls. represents the pumping capacity of this station and also the capacity of the filters, but the arrangements are such that in case of emergency the supply can be increased to 20 000 000 or even 25 000 000 galls. for a short time.

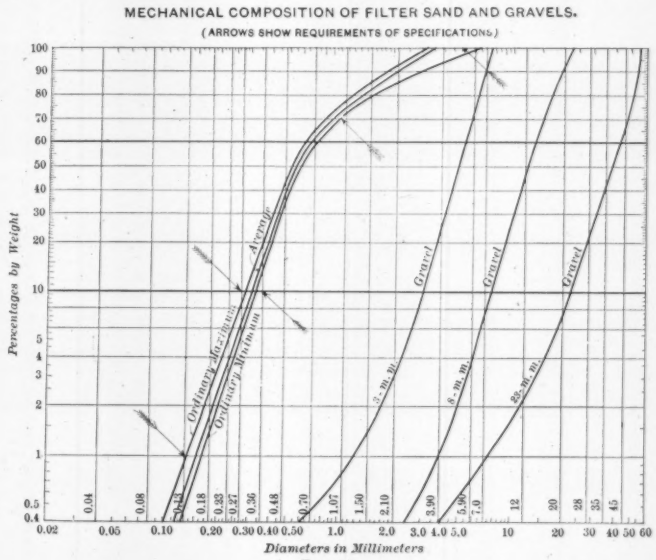


FIG. 14.

The water is pumped through rising mains to reservoirs holding 37 000 000 galls., not including the Tivoli low-service reservoir, which is usually supplied from gravity sources. The reservoir capacity is such that the pumping can be suspended at Quackenbush Street for considerable periods if necessary, and in practice it has been suspended at certain times, especially on Sundays. The amount of water required is also somewhat irregular. The drainage areas supplying the gravity reservoirs are much larger, relatively, than the reservoirs, and at flood periods the volume of the gravity supply is much greater than that

which can be drawn in dry weather. Thus it happens that, at certain seasons of the year, the amount of water to be pumped is but a fraction of the nominal capacity of the pumps, and at these times it is possible to shut the pumps down for greater lengths of time.

Capacity of Pure-Water Reservoir.—The storage capacity provided between the filters and the Quackenbush Street pumps is comparatively small, namely, 600 000 galls., or one hour's supply at the full nominal rate. A larger basin, holding as much as one-third or one-half of a day's supply, would be in many respects desirable in this position, but the conditions were such as to make it practically impossible. The bottom of the reservoir could not be put lower without deepening and increasing greatly the expense of the conduit line. On the other hand, the flow line of the reservoir could not be raised without raising the level of the filters, which was hardly possible upon the site selected. The available depth of the reservoir was thus limited between very narrow bounds, and to secure a large capacity would have necessitated a very large area, and consequently a great expense. Under these circumstances, and especially in view of the abundant storage capacity for filtered water in the distributing reservoirs, it was not deemed necessary to provide a large storage, and only so much was provided as would allow the pumps to be started at the convenience of the engineer, and give a reasonable length of time for the filters to be brought into operation. For this, the pure-water reservoir is ample, but it is not enough to balance any continued fluctuations in the rate of pumping.

Method of Regulating and Changing the Rate of Filtration.—With all the Allis pumps running at their nominal capacity, the quantity of water required will just about equal the nominal capacity of the filters. When only one or two pumps are running, the rate of filtration can be reduced. With the plant operating up to its full capacity, the water level in the pure-water reservoir will be below the level of the standard orifices in the filter outlets. When the rate of pumping is reduced, if no change is made in the gates controlling the filter outlets, the water will gradually rise in the pure-water reservoir and in the various regulator chambers, and will submerge the orifices and gradually reduce the head on the filters, and consequently the rates of filtration, until those rates equal the rate of pumping. In case the pumping is stopped altogether, the filters will keep on delivering at gradually

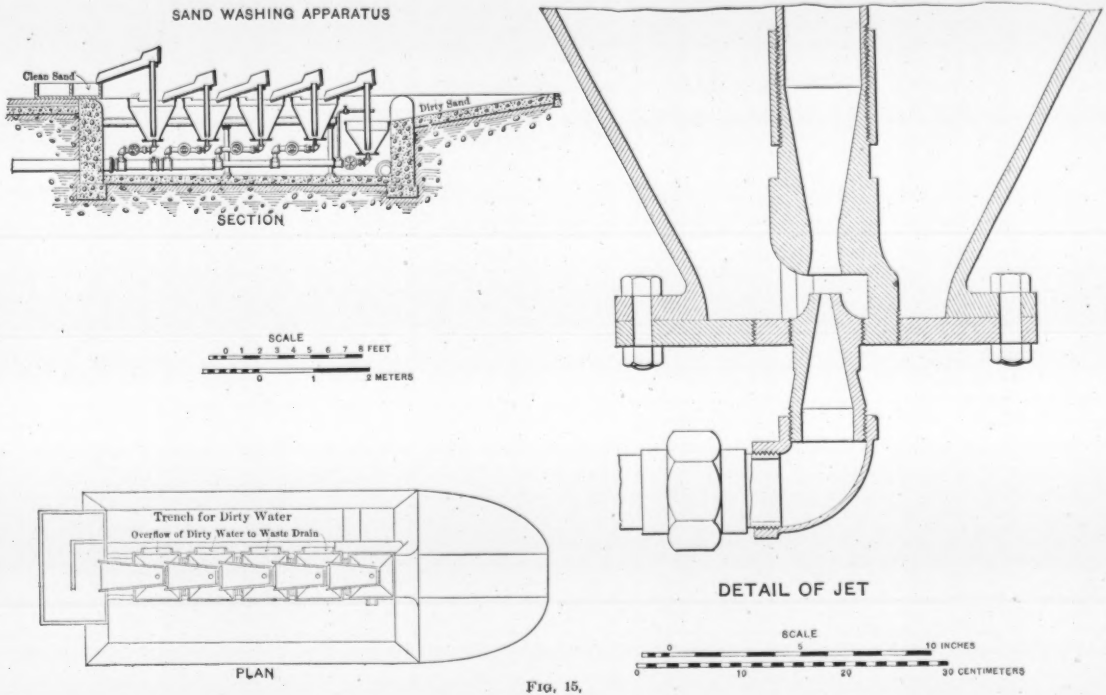

SAND WASHING APPARATUS
Clean Sand
Dirty Sand
SECTION
SCALE
0 1 2 3 4 5 6 7 8 FEET
0 1 2 METERS
Trench for Dirty Water
Overflow of Dirty Water to Waste Drain
PLAN
DETAIL OF JET
SCALE
0 5 10 INCHES
0 10 20 30 CENTIMETERS

FIG. 15.

reduced rates until the water level in the pure-water reservoir reaches that of the water on the filters.

When the pumps are started up, after such stoppage or reduced rate of pumping, the water levels in the pure-water reservoir and in the gate chambers will be lowered gradually, and the filters will start to operate at first with extremely low rates which will increase gradually until the water is depressed below the orifices, when they will again reach the rates at which they were last set. The regulators during all this time will show the rate of filtration on each filter, and, if any inequalities occur which demand correction, the gates on the various outlets can be adjusted accordingly.

The arrangement, in this respect, combines some of the features of the English and German plants. In the English plants the filters are usually connected directly with the clear-water basin, and that in turn with the pumps, and the speed of filtration is required to respond to the speed of the pumps, increasing and decreasing with it, being regulated at all times by the height of water in the pure-water reservoir. This arrangement has been subject to severe criticism, because the rate of filtration fluctuates with the consumption, and especially because the rates of filtration obtained simultaneously in different filters may be different. There was no way to determine at what rate any individual filter was working, and there was always a tendency for a freshly scraped filter to operate much more rapidly than those which had not been scraped for some time.

This led to the procedure, first formulated by the Commission of German Water-Works Engineers in 1894, and provided for in most of the German works built or remodeled since that time, of providing pure-water storage sufficient in amount to make the rate of filtration entirely independent of the operation of the pumps. Each filter was to be controlled by itself, be independent of the others, and deliver its water into a pure-water reservoir lower than itself, so that it could never be affected by back-water, and so large that there would never be a demand for sudden changes in the rate of filtration.

This procedure has given excellent results in the German works; but it leads oftentimes to expensive construction. It involves, in the first place, a much greater loss of head in passing through the works, because the pure-water reservoir must be lower than the filters, and the cost of the pure-water reservoir is increased greatly

because of its large size. The regulation of the filters is put upon the attendants entirely, or upon automatic devices, and regulation by what is known as "responding to the pumps" is eliminated.

More recently, the German authorities have shown less disposition to insist rigidly upon the principles advanced in 1894. In a compilation of the results of several years' experience with German water filters, Dr. Pannwiz,* makes a statement of particular interest, of which a free translation is as follows:

"Most of the German works have sufficient pure-water reservoir capacity to balance the normal fluctuations in consumption, so that the rate of filtration is at least independent of the hourly fluctuations in consumption. Of especial importance is the superficial area of the pure-water reservoir. If it is sufficiently large there is no objection to allowing the water level in it to rise to that of the water upon the filters. With very low rates of consumption during the night the filters may work slowly and even stop, without damage to the sediment layers when the stopping and starting take place slowly and regularly, because of the ample reservoir area.

"The very considerable fluctuations from day to day, especially those arising from unusual and unforeseen occurrences, are not provided for entirely by even very large and well-arranged reservoirs. To provide for these without causing shock, the rate of filtration must be changed carefully and gradually, and the first essential to success is a good regulation apparatus."

"Responding to the pumps" has a great deal to recommend it. It allows the pure-water reservoir to be put at the highest possible level, it reduces to a minimum the loss of head in the plant, and yet provides automatically, and without the slightest trouble on the part of the attendants, for the delivery of the required quantity of water by the filters at all times. If the filters are connected directly to the pumps there is a tendency for the pulsations of the pumps to disturb their operation, which is highly objectionable, even if the pumps are far removed; and this exists where filters are connected directly to the pumps, and a pure-water reservoir is attached to them indirectly. By taking all the water through the pure-water reservoir and having no connection except through it, this condition is absolutely avoided, and the pull on the filters is at all times perfectly steady.

Much has been said as to the effect of variation in the rate of filtration upon the efficiency of filters. Experiments have been made

*"*Arbeiten aus dem Kaiserlichen Gesundheitsamte*," Vol. xiv., p. 260.

at Lawrence and elsewhere which have shown that, as long as the maximum rate does not exceed a proper one, and under reasonable regulations, and with the filter in all respects in good order, no marked decrease in efficiency results from moderate fluctuations in rate. There is probably a greater decrease in efficiency by stopping the filter altogether, especially if it is done suddenly, than by simply reducing the rate. The former sometimes results in loosening air bubbles in the sand, which rise to the surface and cause disturbances, but this is not often caused by simple change in rate.

On the whole, there is little evidence to show that, within reasonable limits, fluctuations in rate are objectionable, or should be excluded entirely, especially in such cases as at Albany, where arrangements to prevent them would have resulted in very greatly increased first cost. The inferior results sometimes obtained with the system of "responding to the pumps" as it existed in earlier works, and still exists in many important places, undoubtedly arises from the fact that there is no means of knowing and controlling the simultaneous rate of filtration in different filters, and that one filter may be filtering two or three times as fast as another, with nothing to indicate it.

This contingency is fully provided for in the Albany plant. The orifices are of such size that even with a filter just scraped and put in service, with the minimum loss of head, with the outlet gate wide open, and with the water level in the pure-water reservoir clear down; that is, with the most unfavorable conditions which could possibly exist, the rate of filtration cannot exceed 5 000 000 or 6 000 000 galls. per acre daily, or double the nominal rate. This rate, while much too high for a filter which has just been cleaned, is not nearly as high as was possible, and in fact actually occurred in the old Stralau filters at Berlin, and in many English works; and, further, such a condition could only occur through the gross negligence of the attendants, because the rate of filtration is indicated clearly at all times by the gauges. These regulating devices have been specially designed to show the rate with unmistakable clearness, so that no attendant, however stupid, can make an error by an incorrect computation from the gauge heights. It is believed that the advantage of clearness by this procedure is much more important than any increased accuracy which might be secured by refinements in the method of computation,

which should take into account variations in the value of the coefficient of discharge, but which would render direct readings impossible.

In designing the Albany plant the object has been to combine the best features of German regulation with the economical and convenient features of the older English system, and filters are allowed to respond to the pumps within certain limits, while guarding against the dangers ordinarily incident thereto.

Results of Operation.

The filters were designed to remove from the water the bacteria which cause disease. They have already reached a bacterial efficiency of over 99%, and it is expected that their use will result in a great reduction in the death rate from water-borne diseases in the city. They also remove a part of the color and all of the suspended matters and turbidity, so that the water is satisfactory in its physical properties.

The filters have reached, with perfect ease, their rated capacity, and on several occasions have been operated to deliver one-third more than this amount; that is to say, at a rate of 4 000 000 galls. per acre, daily.

The filters are operated by George I. Bailey, M. Am. Soc. C. E., Superintendent of the Albany Water-Works, whom the writer hopes will present further particulars concerning the cost and results of operation.

Cost of Construction.

The cost of the filtration plant complete is shown by Table No. 3. As some of the accounts are not yet closed, a few of the items, designated by stars (*) are estimated, and are subject to slight change with the closing of all the accounts.

The filters, sedimentation basin and pure-water reservoir are connected in such a way as to make an exact separation of their costs impossible; but, approximately, the sedimentation basin cost $60 000, the pure-water reservoir $9 000, and the filters $255 000. The sedimentation basin thus cost $4 100 per million gallons capacity; and the filters complete, cost $45 600 per acre of net filtering area, including all piping, office and laboratory building, but exclusive of land and engineering.

TABLE No. 3.—Approximate Cost of Filtration Plant Complete.

Land		$8 290.00
Pumping Station:		
Pumping machinery, boilers, etc.	$22 000.00	
Intake	3 800.00	
Pump well, gates, screens and foundations	6 285.00	
Pumping station building	12 167.00	
Chimney	1 540.00	
Venturi meter	1 560.00	
Extra work and minor items	2 393.00*	
		49 745.00*
Filters, Sedimentation Basin and Pure-Water Reservoir:		
Preliminary draining	$1 956.71	
70 672 cu. yds. excavation, at (average) $0.3079 +	21 761.64	
16 040 cu. yds. rolled clay and gravel embankment, at $0.52	8 340.80	
22 851 cu. yds. silt and loam filling, at $0.15	3 427.65	
23 439 cu. yds. general filling rolled, at $0.18	4 219.02	
12 550 cu. yds. puddle, at $0.715	8 973.25	
1 775 cu. yds. gravel for lining, at $0.85	1,508.75	
2 257 sq. yds. split stone lining, at $0.82	1 850.74	
11 737 cu. yds. concrete in floors, at $2.31	27 112.47	
7 792 cu. yds. concrete in vaulting, at $3.85	29 999.20	
3 147 cu. yds. all other concrete, at $2.13	6 703.11	
4 382 cu. yds. brick work, at $8.125	35 603.75	
31 715 barrels Portland cement, at $1.935	61 368.53	
7 281 cu. yds. filter gravel, at $1.05	7 645.05	
36 488 cu. yds. filter sand, at $1.00	36 488.00	
Cast-iron pipes and specials, placed, including placing gates	21 841.25	
Gates and valves	6 714.23	
Vitrified pipe, complete	7 153.32	
672 filter manhole covers, at $4.40	2 956.80	
8 sand-run fixtures, at $407.50	3 260.00	
8 regulator houses, at $862.24	6 897.92	
1 office and laboratory	4 881.00	
Vitrified brick paving	2 158.00	
Iron fence about court	1 704.00	
Extra work and all minor items	9 692.01	
		324 217.20
Conduit and Connections with Quackenbush Street Pumping Station:		
7 913 ft. 48-in. steel pipe, at $4.50	$35 608.50	
24 218 cu. yds. excavation at (average) $0.855 +	20 715.60	
3 144 cu. yds. concrete, at $5.00	15 720.00	
Gates and connections with pumping station	3 680.00	
Sewer and railroad crossings, sheeting, and all other items	10 914.12*	
		86 638.22*
Engineering, inspection, printing, laboratory equipment, minor construction, etc.		31 000.00*
Total approximate cost of work		$499 890.42*

The preliminary estimate of cost submitted to the Board, February 8th, 1897, amounted to $478 000. The work will actually cost, in round numbers, $22 000, or 4.6% more than the preliminary estimate.

* Estimated.

The preliminary estimate, and the actual costs by more important divisions, are as follows:

	Preliminary estimate, Feb. 8, 1897.	Approximate actual cost.
Land	$10 000	$8 290
Pumping station and intake	34 000	49 745
Filters and sedimentation basin, with piping	327 000	324 217
Pure-water conduit and connection, with Quackenbush Street pumping station	64 000	86 638
Engineering and contingencies	43 000	31 000
Total	$478 000	$499 890

The excess in the cost is in the pumping station and conduit line. The cost of the conduit line was increased by surrounding it with concrete, which was not included in the preliminary estimate, and also by certain changes in the location required by the canal authorities. The pumping station also was made more elaborate and expensive than was contemplated in the preliminary estimate, but the intake was shortened. Otherwise, the work was executed substantially as first planned.

ACKNOWLEDGMENT.

The general plan and location of the plant were first conceived by the Superintendent of Water-Works, George I. Bailey, M. Am. Soc. C. E., and the successful execution is largely due to his efforts. The members of the Water Board, and especially the Construction Committee, have followed the work in detail closely and personally, and their interest and support have been essential factors in the results accomplished. In the designs and specifications for the pure-water conduit the writer is greatly indebted to Emil Kuichling, M. Am. Soc. C. E., and also for most valuable suggestions relative to the performance of this part of the work. To William Wheeler, M. Am. Soc. C. E., of Boston, the writer is indebted for advice upon the vaulting and cross-sections of the walls, and these matters were submitted to him before the plans were put in final shape. All the architectural designs have been supplied by Mr. A. W. Fuller, of Albany. W. B. Fuller, M. Am. Soc. C. E., as Resident Engineer, has been in direct charge of the work, and its success is largely due to his interest in it and the close attention which he and the assistant engineers have given it.

DISCUSSION.

Mr. Bailey. GEORGE I. BAILEY, M. Am. Soc. C. E.—Two filters were put in service July 27th. Four more were started July 28th. All of these ran until August 9th, and three of them continued until August 12th. They were started with the hope of continuing, but the conditions were unfavorable in that the water was pumped direct to the filters, as the sedimentation basin was not ready for use; the court between the filters, in which scraped sand was to be deposited, washed and stored, was neither leveled nor paved; and the water in the river was roiled and disturbed by the contractors' operations, and particularly with the wash-water from the sand being prepared for the remaining two filters. The run of the filters was, therefore, stopped and not again commenced until September 5th, since which time their operation has been continuous.

COST OF OPERATION.

The work was organized as follows:

Filter operation:	10 laborers,	at	$1.50 per day.
	1 foreman	"	2.75 "
	1 watchman	"	1.50 "
	1 chemist	"	1 000.00 per year.
Pumping Station:	3 engineers	"	75.00 per month.
	3 firemen	"	60.00 "

The working day is eight hours for laborers, engineers and firemen, and over-time is paid for at the rates named. Occasionally, extra help has been hired, and paid for at these rates.

The gross cost of operation, including payroll, tools which are still in use, repairs, supplies of all kinds, wash-water, etc., etc., for the period from September 5th to December 25th, inclusive, 118 days, was $6 164.94. In this time 1 470 000 000 galls. were filtered, making an average of $4.19 per million gallons delivered from the filters.

The master mechanic of the works gives the following statement from his records, as the daily cost at the pumping station:

3 engineers	at	$2.48	$7.44
3 firemen	"	1.98	5.94
3 tons coal	"	2.72	8.16
1 laborer	"	1.50	1.50
9 galls. engine oil	"	0.09	0.81
2 galls. cylinder oil	"	0.11	0.22
5 galls. kerosene oil	"	0.10	0.50
5 lbs. waste	"	0.07	0.35
Steam packing, sheet rubber, soap, soda, mops, cloths, etc.			6.58
Total			$31.50

Mr. Bailey.

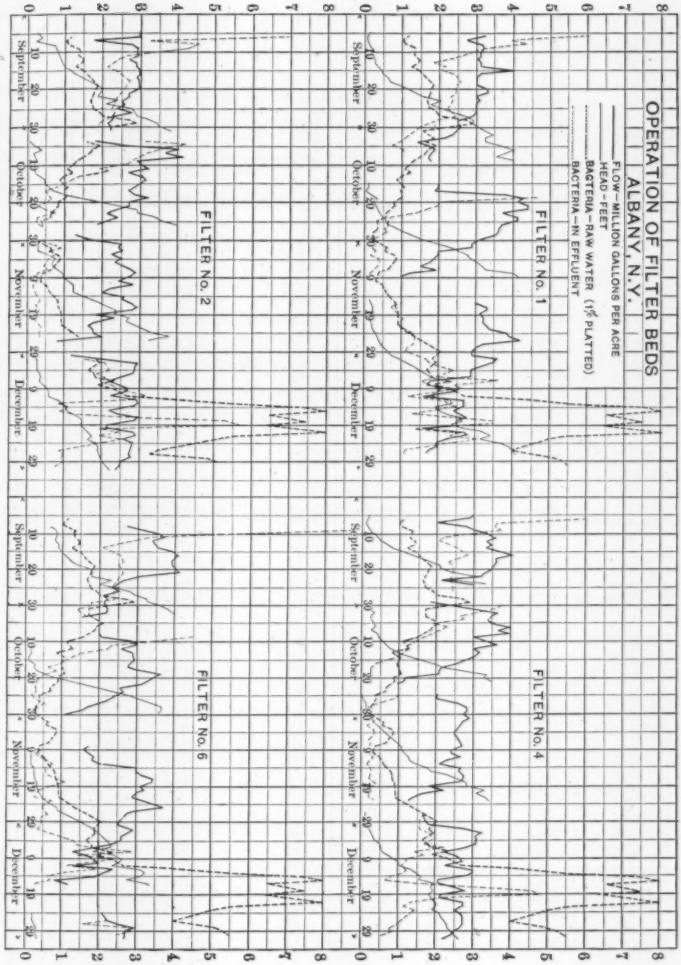


FIG. 16.

Mr. Bailey. This makes the average cost of pumping $2.52 per million gallons received from the filters, and leaves $1.67 as the cost of operating the filters, including laboratory work. The cost of scraping, wheeling out, washing and replacing sand for the actual number of hours, and exclusive of superintendence, laboratory work, lost time, tools, etc., is $1.19 per million gallons treated.

TABLE No. 4.—GALLONS FILTERED AND HOURS EMPLOYED IN SCRAPING FILTERS AND WHEELING OUT SCRAPED SAND.

Filter No.	Service.	Gallons filtered. (5% to be added for error.)	Scraping.		Wheeling Out.	
	Hours.		Hours.	Square yards.	Hours.	Barrows.
1	266.45	22 409 000	43	Each scraping covers an area of 3 402 sq. yds.	86	983
1	791.55	59 164 000	50		80	813
1	559.25	41 457 000	45		81	789
1	1 015.50	72 635 000	42		88	932
2	291.00	26 581 000	46		115	1 027
2	638.00	49 571 000	50		86	962
2	534.00	44 822 000	47		79	771
2	685.00	45 520 000	41		83	886
3	342.75	32 377 000	57		91	1 028
3	639.00	49 296 000	49		75	875
3	579.20	42 296 000	44		82	726
3	877.00	63 406 000	46		92	1 067
4	355.10	32 357 000	52		124	1 119
4	470.20	44 290 000	56		91	942
4	548.00	45 757 000	49		81	763
4	718.50	50 717 000	46		81	878
5	664.60	53 359 000	51		78	820
5	556.70	40 428 000	27		86	874
5	840.00	60 459 000	43		83	802
6	312.70	28 826 000	53		109	1 007
6	618.70	55 263 000	51		78	836
6	547.00	41 133 000	44		76	808
6	932.00	65 457 000	44		88	920
7	631.00	38 506 000	42		69	835
8	361.20	33 006 000	56		105	1 105
8	513.00	40 497 000	53		76	987
8	591.00	50 529 000	50		79	805
8	807.20	56 150 000	40		78	819
28	16 685.60	1 286 268 000	1 317	95 256	2 420	25 179

The filters have been cleaned 26 times in all, up to December 25th, or a little more than three times each. The total amount of sand treated, as measured when replaced, was 850 cu. yds. From the books of the foreman, the following records are taken:

Scraping.—88 452 sq. yds. = 18.3 acres; time, 1 227 hours = 67 hours per acre.

Wheeling Out Scraped Sand.—23 180 barrows, 2 235 hours = 27.3 barrows per cubic yard = 0.38 cu. yd. per hour. The average length of wheel, going and coming, was 600 lin. ft. = 1.18 miles per man per hour.

Washing.—18 262 barrows, 2 068 hours, 21.5 barrows per cubic yard = 0.41 cu. yd. per hour.

From experiments made by John H. Gregory, Jun. Am. Soc. C. E., who was Resident Engineer for Mr. Hazen during the completion of

the work, the speaker is informed that the volume of water for washing the sand varied from 12 to 14 times the volume of sand washed. In the cost of operation the volume has been estimated at 15, at a cost of $0.04 per thousand gallons. Mr. Bailey.

Refilling.—18 550 barrows, 1 630 hours, 21.8 barrows per cubic yard = 0.52 cu. yd. per hour. This work was chiefly done by extra labor. The average depth of scraping was about $\frac{3}{8}$ in., computed from the total quantity of sand replaced and the area scraped.

During the periods covered by these scrapings, the filters yielded 1 212 000 000 galls., an average of 66 600 000 galls. per acre between scrapings. This includes the first run of the filters, when the unnaturally turbid water already mentioned was pumped directly on the beds.

Bacteria.

Table No. 5 shows the weekly averages of the bacterial removal:

TABLE No. 5.

Week ending	Bacteria Per Cubic Centimeter.		Percentage of removal.
	Unfiltered.	Filtered.	
September 9	11 545	608	94.8
16	14 083	306	97.8
23	17 480	273	98.5
30	22 600	259	98.8
October 7	18 766	250	98.7
14	11 783	178	98.5
21	9 933	85	99.1
28	4 733	84	98.4
November 4	6 091	56	99.1
11	5 141	46	99.1
18	7 950	69	99.1
25	11 090	79	99.3
December 2	19 240	109	99.4
9	20 016	198	99.1
16	57 700	142	99.7
23	66 000	327	99.5
30	48 940	215	99.6

Table No. 6 shows in detail the result obtained from each filter each day. It will be noted that the percentage of removal is high, and that the bacterial count in the filtered water is low.

Color.

The average color of the Hudson River water corresponds to 0.50 to 0.60 on the platinum scale, and about 40% of this color is removed from the water by the filters.

Turbidity.

In periods of freshet the water is very turbid. The highest turbidity reached since the operation of the filter was in December, when the raw water showed 0.60. The effluent then contained 0.008. Generally the raw water runs about 0.035, all of which is removed. The platinum-wire standard is used.

Mr. Bailey. TABLE No. 6.—ALBANY FILTER PLANT.—BACTERIAL REMOVAL BY FILTERS.

SEPTEMBER.

Day of month.	Raw Water	1		2		3		4		5		6		7		8		Pure Water.	
	Bacteria.	Bacteria.	Per cent. removed.	Bacteria.	Per cent. removed.	Bacteria.	Per cent. removed.	Bacteria.	Per cent. removed.	Bacteria.	Per cent. removed.	Bacteria.	Per cent. removed.	Bacteria.	Per cent. removed.	Bacteria.	Per cent. removed.	Bacteria.	Per cent. removed.
1	.....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....
2	.....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....
3	.....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....
4	.....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....
5	.....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	800	....
6	11 500	600	94.8	700	93.9	500	95.7	600	94.8	3 700	67.8	.....	....	.....	....	.....	....	800	93.0
7	10 200	392	96.2	331	96.8	397	96.3	361	96.5	1 800	82.4	.....	....	.....	....	420	95.9	508	95.0
8	10 860	432	96.1	457	95.8	363	96.7	355	96.7	913	91.6	1 100	89.9	.....	....	437	96.0	571	94.7
9	13 700	337	96.4	431	96.8	381	97.2	372	97.3	731	94.7	873	93.7	.....	....	541	96.8	567	95.9
10	.....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....
11	13 200	256	98.1	265	97.9	353	97.3	220	98.3	433	96.7	336	97.5	.....	....	194	98.5	420	96.8
12	14 700	231	98.4	243	98.3	320	97.8	207	98.6	440	97.0	289	98.0	.....	....	214	98.5	294	98.9
13	14 300	182	98.7	.....	....	321	97.8	228	98.4	402	97.2	243	98.3	.....	....	220	98.5	284	98.0
14	12 400	218	98.2	211	98.3	.....	....	256	98.0	550	95.6	205	98.4	.....	....	197	98.4	257	97.9
15	14 700	206	98.6	237	98.4	.....	....	273	98.1	362	97.5	228	98.4	.....	....	200	98.6	286	98.1
16	15 200	242	98.4	247	98.4	.....	....	286	98.1	375	97.5	251	98.3	.....	....	216	98.6	298	98.2
17	.....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....
18	17 100	256	98.5	276	97.4	.....	....	232	98.6	360	97.9	261	98.5	.....	....	231	98.7	289	98.3
19	18 900	247	98.7	263	98.6	.....	....	241	98.7	345	98.2	250	98.7	.....	....	221	98.8	273	98.6
20	.....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....
21	17 600	243	98.6	251	98.6	.....	....	249	98.6	336	98.1	261	97.5	.....	....	232	98.7	265	98.5
22	17 100	231	98.6	243	98.6	625	96.3	241	98.6	312	98.2	254	98.6	.....	....	245	98.6	276	98.4
23	16 700	236	98.6	232	98.6	351	97.9	237	98.6	289	98.2	246	98.5	.....	....	231	98.6	261	98.4
24	.....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....
25	17 400	221	98.7	224	98.7	316	98.2	246	98.6	261	98.5	231	98.7	.....	....	219	98.7	250	98.6
26	19 300	231	98.8	237	98.8	320	98.3	240	98.8	275	98.6	239	98.8	.....	....	244	98.7	267	98.6
27	20 200	241	98.8	229	98.9	276	98.6	.....	....	261	98.7	233	98.8	.....	....	237	98.8	257	98.7
28	27 100	197	99.3	205	99.2	267	99.0	.....	....	247	99.1	241	99.1	.....	....	.....	....	261	99.0
29	29 300	172	99.4	191	99.3	240	99.2	.....	....	215	99.3	176	99.4	.....	....	.....	....	244	98.2
30	22 300	258	98.8	189	99.2	236	98.4	375	98.3	237	98.9	179	99.2	.....	....	700	96.9	278	98.8

OCTOBER.

1	17 000	134	99.3	....	....	215	98.7	360	97.9	155	99.1	137	99.2	.....	....	390	97.7	267	98.4
2	.....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....
3	17 200	109	99.4	....	....	194	98.9	265	98.5	....	....	162	99.1	.....	....	420	97.6	230	98.7
4	19 400	98	99.5	313	98.4	180	99.1	278	98.6	....	....	....	....	.....	....	390	98.0	250	98.7
5	21 000	71	99.7	421	98.0	142	99.3	191	99.1	....	....	....	....	.....	....	340	98.4	240	98.9
6	19 200	105	99.5	360	98.2	150	99.2	230	98.8	640	96.7	....	....	.....	....	280	98.6	264	98.7
7	18 800	102	99.5	325	98.3	140	99.3	219	98.8	560	97.0	....	....	.....	....	260	98.6	250	98.7
8	.....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....
9	15 100	....	....	275	98.2	110	99.3	170	98.9	260	98.3	450	97.7	.....	....	220	98.5	239	98.4
10	11 300	....	....	265	97.7	120	98.9	143	98.7	240	97.9	340	97.0	.....	....	156	98.6	233	97.9
11	12 100	....	....	197	98.4	90	99.3	125	99.0	206	98.3	231	98.1	.....	....	144	98.8	170	98.6
12	13 200	....	....	210	98.4	83	99.4	130	99.0	212	98.4	220	98.3	.....	....	170	98.7	178	98.7
13	8 200	....	....	140	98.4	60	99.3	119	98.7	165	98.0	155	98.2	.....	....	110	98.8	135	98.5
14	10 800	....	....	115	98.9	39	99.6	106	99.0	135	98.8	130	98.8	.....	....	87	99.2	115	98.9
15	.....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....
16	9 800	....	....	72	99.3	36	99.6	39	99.6	86	99.1	91	99.1	.....	....	34	99.7	79	99.2
17	10 200	....	....	45	99.6	50	99.5	27	99.7	71	99.3	60	99.4	.....	....	35	99.7	50	99.5
18	10 400	....	....	53	99.5	72	99.3	26	99.8	32	99.7	86	99.2	.....	....	25	99.8	46	99.6
19	11 000	460	95.8	61	99.7	.....	....	24	99.8	74	99.3	54	99.5	.....	....	43	99.6	160	98.5
20	8 800	198	97.8	50	99.4	.....	....	27	99.7	121	98.6	37	99.6	.....	....	41	99.5	76	99.1
21	9 400	200	97.8	24	99.8	.....	....	70	99.3	32	99.7	30	99.7	.....	....	33	99.7	101	98.9
22	.....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....
23	6 900	135	98.1	66	99.0	141	98.0	35	99.5	35	99.5	27	99.6	361	94.8	15	99.8	125	98.2
24	7 000	70	99.0	24	99.7	125	98.2	.....	....	36	99.5	30	99.6	471	93.3	26	99.6	120	98.3
25	3 900	66	98.3	22	99.6	70	98.2	105	97.3	36	99.1	16	99.6	250	93.6	.....	....	70	98.2
26	4 200	69	98.4	33	99.2	63	98.5	87	97.9	30	99.3	26	99.4	125	97.0	.....	....	75	98.2
27	3 500	59	98.3	....	....	64	98.2	36	99.0	24	99.3	54	98.5	100	97.2	64	98.2	58	98.4
28	2 900	62	97.9	....	....	44	98.5	20	99.3	14	99.5	36	98.8	165	94.3	58	98.0	57	98.0
29	.....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....
30	2 500	27	98.9	31	98 8	18	99.3	14	99.4	....	....	17	99.3	150	94.0	16	99.4	37	98.5
31	4 500	24	99.5	68	98.5	26	99.4	34	99.3	....	....	....	....	165	96.3	34	99.3	68	98.5

TABLE No. 6—(*Continued*).

Day of month.	Raw Water	1		2		3		4		5		6		7		8		Pure Water.	
	Bacteria.	Bacteria.	Per cent. removed.	Bacteria.	Per cent. removed.	Bacteria.	Per cent. removed.	Bacteria.	Per cent. removed.	Bacteria.	Per cent. removed.	Bacteria.	Per cent. removed.	Bacteria.	Per cent. removed.	Bacteria.	Per cent. removed.	Bacteria.	Per cent. removed.
November.																			
1	6 350	23	99.6	96	98.5	24	99.6	32	99.5	....	....	....	....	175	97.1	22	99.6	53	99.2
2	5 700	31	99.4	53	99.1	23	99.6	14	99.8	93	98.4	....	....	150	97.4	25	99.6	51	99.1
3	8 500	15	99.8	72	99.2	29	99.7	31	99.6	140	98.4	....	....	L	....	27	99.7	57	99.3
4	9 000	43	99.5	54	99.4	22	99.8	38	99.6	120	98.7	....	....	172	98.1	97	98.9	69	99.2
5	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
6	7 750	34	99.6	43	99.5	19	99.8	22	99.7	61	99.2	....	....	95	98.8	18	99.8	59	99.2
7	6 900	25	99.6	57	99.2	26	99.6	27	99.6	128	98.2	....	....	115	98.3	17	99.8	52	99.3
8	4 000	16	99.6	26	99.4	20	99.5	L	....	90	97.8	....	....	76	98.1	18	99.6	48	98.8
9	3 800	19	99.5	25	99.4	28	99.3	11	99.7	40	99.0	50	98.7	88	97.7	15	99.6	40	99.0
10	3 400	....	....	26	99.2	19	99.5	22	99.4	36	98.9	28	98.2	120	96.5	17	99.5	35	99.0
11	5 000	....	....	16	99.7	14	99.7	33	99.3	62	98.8	35	99.3	104	97.9	15	99.7	44	99.1
12	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
13	6 200	....	....	20	99.7	12	99.8	17	99.7	41	99.3	76	98.8	65	98.9	27	99.6	45	99.3
14	7 350	....	....	26	99.7	17	99.8	25	99.7	38	99.5	57	99.2	89	98.8	35	99.5	57	99.2
15	8 200	.....	.....	33	99.6	29	99.7	28	99.7	45	99.5	55	99.3	96	98.8	37	99.6	65	99.2
16	8 200	98	98.8	25	99.7	....	....	23	99.7	31	99.6	63	99.2	119	98.1	27	99.7	87	98.6
17	8 500	75	99.1	27	99.7	.....	.....	23	99.7	40	99.5	85	99.0	92	98.9	32	99.6	75	99.1
18	9 250	85	99.1	31	99.7	120	98.7	41	99.6	36	99.6	112	98.8	....	....	23	99.8	83	99.2
19	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
20	9 200	107	98.8	15	99.8	135	98.5	21	99.8	28	99.7	40	99.6	....	....	18	99.8	87	99.1
21	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
22	9 500	95	99.0	35	99.6	120	98.7	23	99.8	19	99.8	24	99.8	....	....	17	99.8	85	99.1
23	10 750	115	98.9	36	99.7	105	99.0	40	99.6	28	99.7	73	99.3	....	....	28	99.7	89	99.2
24	12 000	130	98.9	32	99.7	122	99.0	...	....	30	99.8	57	99.5	....	....	32	99.7	101	99.2
25	14 000	140	99.0	32	99.7	150	98.9	...	....	55	99.6	48	99.6	....	....	29	99.8	112	99.2
26	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
27	17 300	135	99.2	....	....	124	99.2	175	99.0	33	99.8	64	99.7	....	....	57	99.8	115	99.3
28	19 400	118	99.4	....	....	108	99.4	160	99.2	35	99.8	44	99.8	....	....	27	99.9	97	99.5
29	21 000	115	99.4	....	....	100	99.5	...	....	25	99.9	50	99.8	....	....	39	99.8	99	99.5
30	.....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....
December.																			
1	18 600	165	99.1	153	99.2	115	99.4	159	99.2	20	99.9	33	99.8	....	....	....	....	107	99.4
2	19 900	179	99.1	210	99.1	185	99.1	187	99.1	56	99.7	64	99.7	....	....	....	....	130	99.3
3	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
4	16 800	245	98.6	223	98.7	256	98.5	195	98.8	110	99.3	136	99.2	....	....	153	99.1	185	98.9
5	18 100	215	98.8	187	98.9	235	98.7	173	99.0	105	99.4	145	99.2	....	....	225	98.8	195	98.2
6	18 500	190	99.0	210	98.9	225	98.8	230	98.8	....	....	185	99.0	....	....	245	98.6	200	98.9
7	23 900	360	98.5	197	99.2	225	99.1	144	99.4	....	....	290	98.8	....	....	265	98.9	219	99.1
8	19 300	245	98.7	128	99.3	212	99.9	172	99.1	....	....	145	99.2	....	....	360	98.2	182	99.1
9	23 500	270	99.2	140	99.4	276	98.8	128	99.5	....	....	246	99.0	....	....	340	98.6	210	99.1
10	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
11	27 000	120	99.6	320	98.8	115	99.6	110	99.6	233	99.1	116	99.6	....	....	310	98.8	152	99.5
12	44 000	213	99.5	285	99.4	117	99.7	128	99.7	249	99.4	198	99.6	....	....	276	99.4	192	99.6
13	55 000	225	99.6	88	99.8	169	99.7	116	99.8	255	99.6	110	99.8	....	....	256	99.5	163	99.7
14	75 000	249	99.7	112	99.9	64	99.9	85	99.9	216	99.7	62	99.9	....	....	211	99.7	96	99.0
15	80 200	185	99.8	87	99.9	182	99.8	68	99.9	308	99.6	72	99.9	....	....	198	99.8	115	99.9
16	65 030	130	99.8	136	99.8	210	99.7	165	99.7	378	99.4	30	99.9	....	....	147	99.8	131	99.8
17	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
18	75 000	350	99.5	525	99.3	450	99.4	460	99.4	575	99.2	....	....	Lo	st	560	99.2	435	99.4
19	64 000	350	99.5	570	99.1	490	99.2	475	99.3	520	99.2	....	....	315	99.5	470	99.3	450	99.3
20	71 000	137	99.8	195	99.7	243	99.7	210	99.7	320	99.5	....	....	312	99.6	350	99.5	275	99.6
21	80 000	268	99.7	250	99.7	325	99.6	135	99.8	360	99.5	....	....	450	99.4	330	99.6	310	99.6
22	56 000	233	99.6	183	99.7	260	99.5	114	99.8	310	99.4	....	....	475	99.2	305	99.5	291	99.5
23	50 000	310	99.4	255	99.5	257	99.5	146	99.7	175	99.6	....	....	Lo	st	150	99.7	205	99.6
24	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
25	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
26	40 000	115	99.7	84	99.8	....	....	103	99.7	117	99.7	225	99.4	315	99.2	140	99.7	162	99.6
27	45 000	110	99.7	98	99.8	.....	.....	95	99.8	100	99.8	160	99.6	340	99.2	90	99.8	180	99.6
28	51 000	....	....	95	99.8	490	99.0	78	99.8	150	99.7	280	99.5	370	99.3	119	99.8	250	99.5
29	53 700	....	....	100	99.8	370	99.4	85	99.8	141	99.7	290	99.5	325	99.4	115	99.8	255	99.5
30	55 000	....	....	80	99.9	380	99.3	55	99.9	112	99.8	250	99.5	280	99.5	125	99.8	230	99.5
31	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....

Typhoid Fever.

Mr. Bailey. The reason for building the filters was the sewage pollution of the Hudson River and the large death rate from typhoid fever. The average number of deaths from this cause for the nine years ending with 1898 was 85 per annum. During the four months in which the filters have been in operation seven deaths from this cause have been reported. For the corresponding months of the nine years ending with 1898 the average number has been 24. The deaths from this cause have thus been reduced in the ratio of 24 to 7, and one of these seven was in a family which did not use city water.

Mr. W. B. Fuller. William B. Fuller, M. Am. Soc. C. E.—It has occurred to the speaker that it would interest the members if a few words were added concerning the details of construction, of the Albany Filter Plant, which present special features.

The sedimentation basin is situated close to the river bank and lies almost wholly in embankment above the natural surface of the ground. The embankments were constructed of the best available materials, and with great care, so as to minimize any effects of after-settling; but, for tightness, reliance was placed on a 16-in. layer of puddle, which covered the entire bottom of the basin and extended up the sides to 1 ft. above high-water level. For a distance of 5 ft. below high-water level, or to below the lowest level to which the basin is likely to be drawn in winter, the puddle is set back 2 ft. 10 ins. from the face so as to be beyond injury from frost, and is covered with a 2-ft. layer of gravel and 10 ins. of blue limestone paving. In all other places the puddle is faced with 6 ins. of concrete, to prevent mechanical injury to its surface and to present an easily cleansed and smooth surface for the interior of the basin.

The materials of the puddle were equal parts of clay and a sandy gravel containing about 40% sand and 60% gravel of all sizes to about 1 in. in diameter. Clean gravel of a size from ½ to 1 in. was tried, but was not a success. These materials were mixed in a screw-paddle, continuous mixer, such as is often used for mixing concrete, water being added and the mixing continued until all the clay lumps were softened and the clay had penetrated thoroughly all the interstices of the gravel.

In this plastic condition the puddle was placed in position in 6-in. layers over a larger surface and left to dry out. In the process of drying, a large number of shrinkage cracks appeared throughout the mass, but by thorough ramming these cracks were closed and the entire mass consolidated. This process was continued for each of the three layers. The idea was, that with the shrinkage cracks closed and the excess water removed, there was no further tendency to shrinkage, and any water then entering the puddle would expand it and close any remaining cracks. Some of this puddle stood uncovered on 1 on 1½

Mr. W. B. Fuller.

slopes for over a year, exposed to rain and frost, but no cracking or deterioration of any kind was noticed, and it remained as hard as hard-pan earth.

The paving of the upper sides of the sedimentation basin is of blue limestone blocks, rather larger than the usual size, being about 10 to 15 ins. deep, 15 to 36 ins. long and 8 to 20 ins. wide. Two masons and one helper together would lay about 16 sq. yds. per day, and the labor cost of laying the stone and gravel, including the teaming of the materials about 800 lin. ft., was $0.72 per square yard.

The gravel used in the joints and under the paving was the waste from the filter-sand screen. It was perfectly clean, of sizes from $\frac{1}{4}$ to to 1 in., the largest amount being about $\frac{3}{8}$ in., and made an ideal material for the purpose.

The piping about the filters and sedimentation basin was all of light-weight cast-iron pipe, with hub and spigot, lead-caulked joints. The entire system was laid at the same time, the trenches and all bell holes being left open, and the joints made water-tight under a hydrostatic pressure of 50 lbs. per square inch, before making any refill.

All the concrete used in the floors, walls and vaulting, amounting to 22 100 cu. yds., was machine-mixed, especial care being taken with the mixing and placing. A mixed sand and gravel was obtained from the river by dredging, and was brought to and deposited near the mixers, and then washed and screened into three sizes—sand, gravel and tailings. The sand was of very good quality, sharp and clean. The gravel was of irregular, smooth-edged stone from $\frac{1}{4}$ to $1\frac{1}{2}$ ins. in diameter, but varying greatly as to average size from day to day, sometimes fine and sometimes coarse. The tailings were passed through a stone crusher and broken to sizes ranging from $\frac{3}{4}$ to $2\frac{1}{2}$ ins. in diameter.

Mechanical analyses of the sizes of these three materials were made daily or oftener, and from these analyses the proper proportions of a mixture of the three to give a minimum number of voids was deduced. The total quantity of the three materials used with a unit quantity of cement was always constant, but by thus varying the proportions of the ingredients themselves, a very uniform concrete product was obtained, independent of the variation in the average size of any particular ingredient. The proportions ordinarily followed were 1 part of cement, 3 parts of sand, 4 parts of gravel, and 1 part of crushed stone.

The mixing was done in a cubical mixer, to which a measured quantity of water could be introduced after the materials had been thoroughly mixed dry. With the apparatus as used at Albany, the concrete was always mixed properly and of the right consistency; half an hour's attention to it, when starting, insuring uniformity for all day unless there were great weather changes, in which case the quantity of water had to be changed more frequently.

Mr. W. B. Fuller.

The transportation of the concrete from the mixer and its deposition in place in the work, which has been described by the author, was ideal, from an engineering point of view. From the time of taking the concrete from the mixer until it was put in place 800 ft. distant the interval of 5 minutes was not uncommon, and the average time did not exceed 15 minutes.

The cost of labor, coal, and for measuring quantities, and mixing, loading, transporting and tamping concrete, during an average of about three months of the best organization, was about as follows:

Measuring, mixing and loading....	$0.20	per cubic yard.
Transporting by rail and cables....	0.12	" "
Laying and tamping on vaulting...	0.14	" "
Laying and tamping floors and walls, including setting forms...	0.22	" "

These prices do no not include general superintendence, profit or cost of machinery.

The design of the vaulting, as has been stated by the author, is in the form of an elliptical groin with dimensions as follows: span, 12 ft.; rise, 2 ft. 6 ins.; thickness at crown, 6 ins.; thickness over center of pier, 2 ft. 6 ins. This work was figured for strength according to the theory of the arch, assuming that brick masonry and concrete spandrel filling were to be used. The material actually used was Portland cement concrete laid as a monolith, with the pier in the center, and, as thus constructed, it is exceedingly doubtful if there is any arch action whatever in the structure. From recent observations and from some tests made on small models the speaker believes that such a groined arch acts in tension as an inverted pyramidical dome. If this is the case, the depression over the piers could be increased materially and the cost of the vaulting reduced. Even as constructed, the adoption of the 6-in. depression over each pier saved 1 053 cu. yds. of concrete, which would have cost $6 560 at contract prices.

The speaker wishes to controvert an impression, which seems to be prevalent, that permanent masonry vaulting is very expensive, several recent estimates placing the cost of covered filters at from 50% to 100% in excess of the cost of open filters. The total extra cost of the vaulting at Albany, including extra thickness of floors, piers, roof drains, manholes, sand-run entrances, earth covering, etc., at the contract prices, was approximately $0.315 per square foot of area inside the walls, or $13 700 per acre, while the total cost of the filters was about $45 600 per acre; that is, the vaulting represents only about 30% of the cost of the filters.

The lumber of the centering for the vaulting was of spruce for the ribs and posts, and of hemlock for the lagging, which was 1½ and 3 ins. wide and 1 in. thick. The entire centering for two filters was formed by machinery, the ribs put together to a template, and the lagging all

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sawed to proper lengths and bevels. For the first two filters the centers were put together in place, and were so constructed that when struck they would come down in sections and could be moved forward and used in the corresponding bay of the next filter.

The total cost of the centering was approximately as follows:

Building centers covering 62 560 sq. ft.

Labor—					
Foreman,	435 hours,	at	$0.35	$ 152.25	
Carpenters,	4 873 "		0.225	1 096.42	
Laborers,	3 447 "		0.15	517.05	
Painters,	577 "		0.15	86.55	
Teaming,	324 "		0.40	121.60	
				$1 973.87	
Materials—					
Lumber,	313 000 ft., B. M.		$5 700.00		
Nails,	3 700 lbs		111.00		
Tar,	8 bbls		24.00		
				5 835.00	
					$7 808.87

Taking down, moving and putting up centers covering 196 660 sq. ft.

Labor—					
Foreman,	2 359 hours,	at	$0.35	$ 825.65	
Carpenters,	12 766 "		0.225	2 872.35	
Laborers,	24 062 "		0.15	3 609.30	
Team,	430 "		0.40	172.00	
				$7 479.30	
Materials—					
Lumber,	3 000 ft., B. M.		$60.00		
Nails,	3 000 lbs		90.00		
				$150.00	
Total				$7 629.30	
					7 629.30
Total approximate cost					$15 438.17

The total area centered was 259 220 sq. ft., and the average cost per square foot was 6 cents, to which should be added general superintendence and a reasonable profit.

As bearing upon the need of a covering, for protection against frost, the following records for Albany are quoted. The temperature of the air varies from + 98° to — 18° Fahr. per annum, the average temperature for the year being + 45° Fahr. These figures are the result of 22 years' observation by the local weather bureau. The river is frozen over from December 16th to March 19th, an average length

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of 93 days, as shown by an average of the records of 75 years. The longest time in any one season in which the river has remained frozen over was 117 days.

There are other advantages, besides the prevention of the formation of ice, which enable the covered filter to be operated at a lower cost for maintenance than the open filter, a few of which are as follows:

By preventing the formation of algæ growths.

By maintaining the temperature of the water practically even, thus keeping the friction of the sand constant, as by maintaining uniform conditions, improvement of the filtrate results.

By preventing the action of the heat and the direct rays of the sun in summer from baking the surface during the operation of scraping, thereby occasioning the removal of a greater thickness of sand.

By keeping the surface from disturbance by wind and thus allowing more efficient sedimentation.

By preventing the fall of rain on the surface during cleaning, which causes compacting of the surface and necessitates re-raking.

By preventing the fall of snow on the surface during cleaning, which causes the removal of a greater thickness of sand.

By preventing the increase in cost of washing sand containing leaves and algæ growths.

By admitting of uninterrupted scraping of the filter during all kinds of weather.

As the extra cost of vaulting is only about $13 700 per acre, which by added knowledge can possibly be reduced to $10 000 per acre in another plant, and which amount at 3% interest represents only $300 to $400 per year, it is seen that in many places other considerations besides that of protection from ice would lead to the use of covered filter beds.

Mr. Maignen.

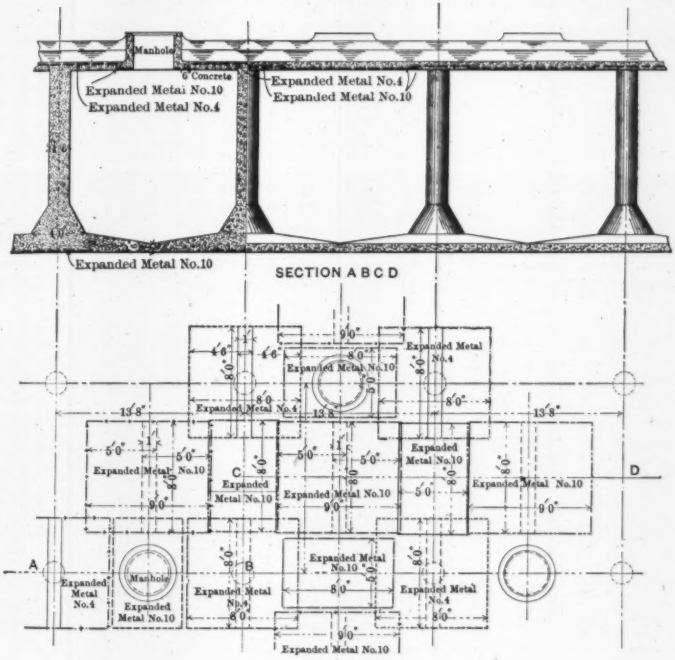
P. A. MAIGNEN, Assoc. Am. Soc. C. E.—The construction of the filters at Choisy-le-Roi, near Paris, is interesting. Some of the walls of the filters are very thin, being only 2.4 ins.; and their construction with what is termed in the United States "expanded metal" and concrete has been quite satisfactory. The company which operates these filters has built filter beds 120 ft. square, and others 60 ft. square. They found the smaller beds better for solidity and also because of the greater convenience in cleaning.

In the Albany plant the sand-washing apparatus is not covered. The speaker supposes that it is the intention to have it covered, in order that the washing of the sand may not be delayed by freezing weather.

The provision made for preventing the raw water, which may go down through or along the retaining walls, from mixing in an unpurified state with the filtered water, is interesting. It would seem better that such provision be made at the top of the sand bed instead of at the bottom.

Mr. Hill. GEORGE HILL, M. Am. Soc. C. E.—The paper is so completely what it should be, it describes so completely the way in which the work was done, its cost and its effect, that the speaker does not wish to be understood as offering any criticism in calling attention to a few details wherein the cost of the work might have been reduced, without in any way impairing its efficiency.

Referring to Fig. 7, it might be well to note that 8-in. beams are too shallow to be used with a projection of 5 ft., as they would be apt to



PLAN SHOWING DISTBIBUTION OF EXPANDED METAL

FIG. 17.

deflect so much as to crack the concrete, admitting moisture and therefore hastening decay. Fifteen-inch beams used in the grillage would have had five times the strength, with but two and one-half times the weight, would have had practically no deflection, a thicker section to resist rust, and, being spaced wider, would have given more room to pack the concrete between them.

The steel track used by the speaker, and described in a previous

Mr. Hill. paper,* would have been particularly applicable for the handling of dirty sand, as mentioned on page 264, and would have been found to be inexpensive. The speaker has used it for coal and ash tracks carrying loads of 500 lbs. very satisfactorily.

The principal feature wherein a saving could have been effected was in the floor and covering of the filtration basins. At the time when estimates were being made, one of the prospective bidders consulted with the speaker regarding the construction proposed and was advised to suggest the plan shown in Fig. 17.

It is probable that the lack of any published data regarding the action of steel and concrete in combination, and especially the lack of knowledge regarding the resistance of continuous slabs supported at a number of points, lead, in a measure, to the rejection of this advice, if it ever was presented by the bidder. Numerous experiments made by the speaker confirm him in the belief that the carrying capacity of a slab supported at a number of points and made continuous over them, is, for a portion of the slab included between one set of supports, four times as great as that of a slab of the same sectional area and dimensions supported at two opposite edges, or, for the sections proposed, 650 lbs. per square foot, safe working load.

The relative costs for one bay are as follows:

As executed:	
5.8 cu. yds. vaulting	$22.35
8.72 " flooring	20.15
1.24 " brick work	10.08
Total	$52.58
As proposed:	
10 cu. yds. in roof slab, supporting column, floor and foundation	$23.10
Centering	7.47
Expanded metal	10.60
Total	$41.17

Saving 22 per cent.

It will be observed that the centering is far more simple, and the placing of the concrete less costly. The ramming of the concrete could be done with a roller instead of by hand, thus effecting a further saving. None of these points has been taken into account in the above comparison. The capacity of the filter could have been increased by raising the upper limit of the filling, or the cost decreased by reducing the height of the pier and decreasing the depth of the excavation.

Transactions, Am. Soc. C. E., Vol. xxxix, p. 620.

A. M. MILLER, M. Am. Soc. C. E.—There appears to be some misapprehension as to the cost of concrete. The author states the price per cubic yard paid to the contractor, but the City of Albany furnished the cement. The cost mentioned by Mr. Hill, for the construction of the filters, is exclusive of the cost of cement. The author calculates that 1¼ bbls. of cement per cubic yard were required, and this, at 1.93½ per barrel, would add $2.42 to the cost per cubic yard. This must be borne in mind when examining the estimates. These questions, however, as to details of costs, strength of materials, etc., can be readily answered. Mr. Miller.

The speaker has heard that the Board of Health of New York City has stated that eventually the whole of the Croton water supply, some 300 000 000 galls. per day, must be filtered. The handling of such a large quantity will be a very serious question. The speaker is now considering the filtration of a supply of 60 000 000 galls. per day, and is glad to listen to those who have had some experience in these matters.

Information is needed as to methods of examining water, the quantity or thickness of sand required for filtering, the rate of filtration, etc., and Mr. Hazen's paper is very instructive in regard to these matters.

Mr. Bailey's statements in reference to the percentage of bacteria removed are interesting, though the speaker does not believe in expressing the results in that way. For instance, even though 95% of the bacteria may be removed by filtration, yet, possibly 700 bacteria come through, and thus the percentage method is misleading.

The cost of the filters per acre is an important item. From the statements of cost in the paper it is found that this was about $45 600. In the recent report of the Board of Experts on the water supply of Philadelphia, the cost is stated as about $37 000 per acre. Is the difference in cost due to the use of covered and open filters? If so, what is the estimated cost of the vaulting per acre?

RUDOLPH HERING, M. Am. Soc. C. E.—Albany has a raw water supply, as well as the filtered supply, and these two supplies are mixed and stored in an open reservoir. Mr. Bailey's statement in reference to the reduction of the typhoid fever death rate applies to the mixed waters. If this death rate should not be reduced as much as in other cities the filter should not be debited with the discrepancy, because raw water is added to the filtered water, and because both are stored in an open reservoir exposed to the air, dust, and other aerial contamination. Mr. Hering.

In Philadelphia it is proposed to cover all filters and reservoirs containing filtered water. The difference between the cost of the Albany filters and the estimated cost of the Philadelphia filters, is due to the difference in the prices of some of the materials, and some parts of the design. Otherwise there is no difference.

Mr. Hering. Mr. W. B. Fuller's description of the details of construction is interesting and valuable. He has pointed out very clearly the advantages of having covered, rather than open, filters, and the probable additional cost thereof. He has also pointed out the advantage of a batter on the sides to prevent the formation of seams which would allow raw water to pass through the filter more rapidly than permissible. An expedient which further helps to prevent an undue speed of the percolating water through the larger interstices necessarily formed between the sand and the smooth surface of the adjoining wall, is the transformation of this smooth surface into a sanded surface. This is done simply by first painting it with cement and then throwing sand against it. The sand, which sticks when the cement hardens, prevents the formation of interstices of excessive size.

Dr. Mason. Dr. WILLIAM P. MASON.—In the removal of the taste or odor produced by the discharges from gas-works, the Albany plant has failed, and in this, any plant must, of necessity, fail. There have been many complaints in Albany in relation to this matter, and the best way out of the difficulty seems to be to arrange with the gas company not to put any of their waste products into the water.

TABLE No. 7.—COMPARISON OF COSTS OF OPERATION OF FILTERS AT ALBANY, N. Y., WITH THOSE IN SEVERAL EUROPEAN CITIES.

Costs are based upon an 8-hour day at $1.50.

Location of Filters.	Items.	Time in man-hours per acre.	Cost.	
			Per million gallons.	Per acre.
Albany.....	Scraping dirty sand into piles.....	61.1	$0.19½	$11.45
	Wheeling out dirty sand and leveling beds.....	111.0	0.35½	20.81
	Washing sand, wheeling it back to filter and leveling the top: $1.13½ — (0.19½ + 0.35½) = $0.58½	.....	0.58½	.....
	Total cost of cleaning.....	.....	1.13½	32.26
Liverpool....	Scraping. (Labor, $0.91 for 10 hours per day).....	.....	.....	12.50 to 25.00
London..... (New River.)	Scraping and wheeling out dirty sand, costs $13.39 per acre, with labor at $0.93 for 10 hours, or, based on an 8-hour day at $1.50.....	144.0	.....	27.00
London..... (Southwark and Vauxhall.)	Scraping and wheeling sand to sand-washer costs $38.00 per acre, with labor at $0.95 for 10 hours, or, based on an 8-hour day at $1.50.....	400.0	.....	75.00
Schiedam....	Scraping and wheeling out dirty sand costs $10.44 per acre, with labor at $0.60 for 10 hours, or, based on an 8-hour day at $1.50	174.0	.....	32.62
Rotterdam...	The mean cost for filter management during 10 years has been.....	.....	$1.53	.....

Dr. Mason. The speaker, being interested in the subject, has made some experiments in reference to this flavor or odor. The English plan of filtration was a failure, likewise the mechanical filters; but the speaker has found that, after the water had been treated in a mechanical plant, the difficulty was overcome by passing the water through carbon. The simplest way, however, is to prevent the gas material from entering the water.

The figures in Table No. 7, in reference to the operation of filters, have been arranged in such a manner as to facilitate comparison with the cost of operating some of the European filter plants.

Mr. Fowler. CHARLES E. FOWLER, Esq.* (by letter).—The writer has had no experience in the operation of covered filters, and, therefore, cannot be certain that there are not difficulties in their operation unknown to him. Judging, however, from his experience in the use of open filters, it would appear that the usual hindrances to successful operation had been entirely removed in the construction of the Albany plant. Indeed, it would seem that the only difficulty to be apprehended in the operation of the plant would be the tendency to place undue reliance upon the automatic appliances which, however perfect and complete, require watching. To one accustomed to battle with atmospheric conditions in operating an open filter plant in this latitude, the supervision of such a plant as described would seem a pleasant pastime.

It is ordinarily understood that the chief object to be attained in covering a filter is to obviate the difficulties and expense arising from the action of frost, and this possibly may be the chief, but it is by no means the only, object to be sought, at least in dealing with a water having the characteristics of the Hudson River water at Poughkeepsie. The algæ growths on the sand in summer are quite as troublesome and almost as expensive as ice and frost in winter. Like ice, they can develop on an unlimited area in the same time as on a small unit, and will stop a filter and put it out of service just when it should be otherwise doing its best work. Covering, as described in this paper, will prevent the development of algæ entirely. During the summer of 1899 the filters at Poughkeepsie were scraped six times, from May 1st to October 10th, at a cost of about $70 for each scraping. Had it not been for algæ growths, three scrapings in that period would have been sufficient. The cost of the extra scrapings necessitated by the algæ growths, together with the cost of washing and replacing the additional sand removed, amounted in 1899 to more than the cost of removing ice.

There is still another evil incident to an open filter in the hot months. It is essential to the proper working of a filter that, in scraping, as little sand as possible be removed, and that the depth removed

* Superintendent and Engineer of Public Works, Poughkeepsie, N. Y.

Mr. Fowler. be as nearly uniform over the entire area as possible, about $\frac{1}{2}$ in. being the ordinary depth. In the hot months, after a filter has been in use for two or three years, the surface of the sand, when exposed for scraping, and thus subjected to the rays of the sun, will bake, in the course of a few hours, to a depth of 1 or $1\frac{1}{2}$ ins., or even, in extreme cases, 2 ins. It frequently happens, therefore, that in commencing to scrape a bed in the morning the normal thickness will be removed, but before the work is finished, on account of the action of the sun on the exposed surface, it may be necessary to remove three or four times the normal thickness, thus lessening the efficiency of the bed and largely increasing the expense of scraping. The writer, therefore, from his experience, would urge the covering of filters in any climate, particularly for river waters similar to those of the Hudson.

From the writer's experience it is essential that a filter should always be filled, to a short distance above the sand surface, from below. It seems impossible to admit a current of water on the dry sand so as not to wash or furrow it, and even after filling, the escape of the confined air greatly disturbs the uniform compactness. The writer, therefore, notes with pleasure the completeness of the arrangement for filling the Albany filters from below.

The writer is of the opinion that the single filters of the Albany plant have as great an area as can be operated with economy and efficiency. However great the plant to be installed, it should be composed of units no greater in area than those described. Units of lesser area may be used where desired; indeed, it is the writer's opinion that no plant, however small, should be composed of less than four distinct units of equal area, combined as in the plant under consideration.

The quantity of gravel used appears to have been reduced to a minimum. This is rendered possible by the concave form of the bottom. It may, however, be questioned if the depth over the collecting pipes, $4\frac{1}{2}$ ins., is sufficient to insure, for the greatest period practicable, against sand reaching the collectors and obstructing them. When the old filter at Poughkeepsie, designed by the late James P. Kirkwood, M. Am. Soc. C. E., was opened for repairs in 1897, after having been in operation twenty-five years, sand was found all the way down to the bottom of the filter, although the original depth of stones and gravel was 4 ft.

The depth of broken stones was 2 ft. Above the stones were four courses of gravel, each 6 ins. in thickness. In the absence of actual measurement, it is believed that the mean effective size of the gravel in the upper course was but little if any greater than that in the upper course of the Albany filters, though it was evidently less carefully screened. One of the causes tending to force the sand down into the gravel was, undoubtedly, the invariable practice of filling the bed

from the surface,—there being no other means provided. The avoidance of this practice, in the Albany filters, together with the greater care in screening the gravel, will lessen the tendency of the sand to pass downward; nevertheless, the writer believes that some sand will, in time, however remote, reach the collectors, and that an additional thickness of gravel would prolong that time in a ratio greater than that of the increased thickness. It seems to the writer that about double the thickness of each grade of gravel over the top of the conductors would remove the time of possible obstruction of the collectors to a period in the future sufficiently remote to justify the additional expense of construction. Mr. Fowler.

The device for aeration, numerous small holes near the top of the vertical extension of the inlet pipes to the settling basin, would seem to require the addition of some appliance for keeping the holes clear and preventing their becoming closed by algæ and floating or suspended matter.

The devices for preventing undue loss of head or pressure upon the filters, for observing and regulating the rate of filtration, and for preventing an excessive rate, appear to be admirably adapted for their respective purposes.

The writer congratulates the author and all associated with him in the design and construction, as well as the City of Albany, upon the successful installation of a filtering plant apparently so nearly perfect in all its arrangements for successful operation.

George W. Fuller, Assoc. M. Am. Soc. C. E. (by letter).—This important paper, dealing with the largest plant of its kind now in operation in this country, is a valuable contribution to the subject of water purification. In many ways this plant shows the results of careful thought and of thorough studies of the experimental evidence obtained in this country, and also of the construction and operation of municipal plants in Europe. Mr. G. W. Fuller.

Comparing the Albany plant with those constructed years ago in Europe for the purification of the general type of water of which the Hudson River is representative, it is found that this plant contains many improvements. It is thoroughly modern, embodying the results of the progress of the last dozen years in this particular line. For many years the construction and results of operation of the Albany plant will doubtless be studied by engineers interested in water purification.

In reading this very interesting paper there occurred to the writer a number of points of inquiry and comment, the principal of which are as follows:

Character of Hudson River Water with Reference to Turbidity and Color.—The paper deals at considerable length with the sewage pollution of the unfiltered water, but leaves the reader to his general infor-

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mation with regard to the nature, degree and duration of turbid water which occurs in the Hudson at Albany. Obviously, this factor was of some significance in the design of the plant, for the reason that it was decided to construct a sedimentation basin. All members of this Society are doubtless familiar with the general facts that the Hudson flows through glacial drift formation and not through a clay-bearing region; that the headwaters of the river are in a mountainous country, clad in the winter with snow which melts rapidly in the early spring so as to produce freshets; and that at Albany the river is a comparatively short one. In the near future, specific information will presumably be available on this subject; but the writer desires to inquire what the general evidence now available shows as to the amount and character of suspended matters in the water during freshets; and also the frequency, intensity and duration of freshets.

It is stated at the close of the paper that the filtered water is satisfactory in appearance regarding color, although it is not stated how much color is found in either the unfiltered or filtered water. The question of how much color due to dissolved organic matters may be present in a water of satisfactory appearance is one upon which there is much difference of opinion. And it is of especial significance in connection with the type of plant adopted at Albany, because, ordinarily, it is possible by this means to remove only about one-third of the color of the applied water. While satisfactory results were doubtless obtained in this particular instance, this is, nevertheless, an interesting topic for discussion.

Filtering Materials and Underdrains.—In connection with the efficiency of the filter, the filtering materials and underdrains are of prime importance. The Albany plant, in general terms, represents the best modern theory and practice in these particulars, and it is here that this plant differs most from the older ones in Europe. Formerly it was the practice to use gravel layers much thicker than was necessary to support the sand and conduct the filtered water to the collecting pipes, and, in some instances at least, the layers were not properly graded to prevent the upper layers from settling into the lower ones. At the present time many reliable data are available to serve in making these computations. Concerning the use of the thin graded layers of gravel, there is no doubt that they conduce to both efficiency and economy. In the case of the Albany plant they appear to be worked out as finely as it is safe to adopt in practice. In fact, in the opinion of some engineers, it might be considered questionable whether, with only three grades of gravel, it is advisable to use with the finer grades such thin layers as 2 to 2.5 ins. The experience of the writer shows that ordinarily no difficulty should arise under these conditions, but that rigid inspection of the work is necessary to guard against the layers settling together. The writer desires to inquire if

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any evidences have been detected of the sand passing into and through the gravel layers.

The practice of placing no gravel layers beneath the sand within a distance of 2 ft. 4 ins. of the walls of the filter is especially commendable. If this had been adopted in the older filters it would have doubtless precluded many instances of unfiltered water reaching the filtered water drains.

Filter Covers.—Although there are a number of open filters in service in the general section of the country in which Albany is located, the question of the advisability of covering these filters is too obvious to be a fitting one for discussion. The principal point, in this connection, is to note that for the first time in this country the vaulting for filter covers has been made entirely of concrete. This departure results in economy, and doubtless will be adopted in many instances in the future.

Pure-Water Reservoir and Control of Rate of Filtration.—The description and discussion of these phases of the plant are among the most interesting parts of the paper. They are unusual and unique in a number of ways, and, while not such as to be a model type under many conditions elsewhere, they appear to serve their purpose admirably in this case and to be based on sound reasoning.

Cost of Filters.—The cost of these covered filters, $45 600 per acre, exclusive of land and engineering, is much less than the general figures obtained from other and earlier plants. While there is no doubt that efficient and durable filters of this type can be built, ordinarily, much more cheaply than has been generally considered to be the case, yet the conditions for construction at Albany were usually favorable in a number of ways, as follows:

1. The filter site was a level tract of land, requiring practically no grading and no excavation other than that necessary to obtain material for the embankments.

2. Very little rock excavation was required.

3. The floor of the filters rests on blue or yellow clay, as compared with the quicksand and the made land which would be encountered in some localities.

4. The conditions, apparently, were free from complications, and required no very expensive steps relative to the exclusion of ground-water from the plant. Concerning the leakage of the filtered water, in the event of cracks in the fairly light masonry, the ground-water level would cause the loss of water to be very small, compared with conditions found elsewhere.

5. Construction materials were much cheaper at the time the Albany contracts were let, than at present.

6. The site was very favorably located with reference to securing and delivering the various materials of construction.

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7. With open filters, heavier masonry in some respects would be required than in the case of covered filters.

In noting these points, in regard to which Albany was very fortunate, it is, of course, obvious that they do not detract from the merit of this paper on a plant which, in the writer's opinion, is entitled to great praise.

Mr. Whipple.

GEORGE C. WHIPPLE, Assoc. M. Am. Soc. C. E. (by letter).—The writer wishes to express his appreciation of the service which the Water Department of Albany has rendered to the public in the construction of the filter, so well described by Mr. Hazen. Not only will this filter prove a blessing to the citizens of Albany by the saving of lives, but it will stand as a model for American engineers and an object lesson to certain American cities which have been negligent in protecting themselves from the dangers of their polluted water supplies.

The brief period during which the Albany filter has been in operation is not sufficient to show exactly the degree of purification of the Hudson River water which will be attained permanently, but the figures presented by Mr. Bailey show that already good results are being obtained, and that the bacterial efficiency is improving steadily.

On the other hand, it has been found that there are certain things which the filter will not do. It will not remove all the coloring matter from the water. Experts realize that this is not to be expected, but the ordinary consumer does not understand why the filtered water should not be colorless. Experiments have shown that simple sand filtration is not capable of removing more than about one-half of the coloring matter from water, under favorable conditions, and that ordinarily the amount of reduction is not more than one-third or one-fourth. Mr. Bailey has stated that the color reduction at Albany thus far has been about 40%, the color of the applied water being at times as high as 0.50 or 0.60 on the platinum scale. The removal of color by a new filter is usually greater than by one which has been long in use, and it is probable that this percentage of removal cannot be always maintained. The removal of coloring matter from the Hudson River water by the sand filters at Poughkeepsie is shown by the following figures taken from analyses made at various times by Dr. T. M. Drown:

	Feb., 1891.	Nov., 1891.	Dec., 1891.	Jan., 1892.	Apr., 1894.
Color of applied water	0.233	0.15	0.60	0.38	0.30
Color of filtered water	0.19	0.10	0.65	0.40	0.25
Percentage of reduction of color	17	33	....	....	17

In the spring of 1899, the writer obtained the following results from the Poughkeepsie filter:

Color of Hudson River water	0.32
Color of tap water in Poughkeepsie	0.26
Percentage of removal of color	19

Mr. Whipple.

It has been found at Albany that sand filtration is not always effective in removing certain odors from the applied water. Professor Mason has alluded to the odor imparted to the water of the Back Channel by waste material discharged from the gas-works, and which, not being removed by the filter, caused some complaint from the consumers. This matter of odor, in water which is to be purified, should not be overlooked by engineers when considering filtration works. Inasmuch as sand filtration cannot be depended upon to remove such odors as those observed at Albany, it is essential that the applied water should be freed from them, and it is a case where prevention is easier than cure.

Many American streams receive large amounts of factory refuse, and in such cases filtration alone may not always render the water palatable. The refuse should be purified before it is allowed to enter the streams. Sometimes very large streams become affected with odors from factory wastes. The Schuylkill River at Philadelphia is a case in point. A few years ago the water in certain sections of Philadelphia acquired a disagreeable odor which resembled that of creosote as much as anything. Through the courtesy of John C. Trautwine, Jr., Assoc. Am. Soc. C. E., the writer was given the opportunity to make some observations as to the cause of this odor. Samples of water were collected in various parts of the city and examined physically and microscopically. The character of the odor showed that it was not due to microscopic organisms, and no odor-producing organisms were found in the water. The odor was apparently due to manufacturing waste, and suspicion fell upon certain paper mills a few miles above the city from which large volumes of refuse material were discharged at certain times during the day. Samples of water collected from the river had the same odor as that observed in the city, and near the mill where the refuse material was supposed to be discharged the odor was very decided. The microscopical examinations of the samples showed the presence of fragments of wood fiber in the river water and in some of the samples of tap water. The samples which contained the largest amounts of wood fiber gave in general the strongest odors. Furthermore, it was the testimony of many individuals that the odor was not of equal intensity throughout the day, this being due, apparently, to intermittent discharges from the mills.

There is one feature of the operation of the Albany filter which will be watched with interest by biologists, namely, the storage of the filtered water in an open reservoir. It has become a well-recognized principle in this country that ground-waters cannot be stored in reservoirs exposed to the light without liability to deterioration from troublesome algæ growths; and the question has already presented itself to the minds of some as to whether the water supply at Albany will not some day suffer from this cause. Time alone will tell whether

Mr. Whipple. their fears will be realized. That diatoms and other microscopic organisms will develop to some extent in the stored water is to be expected, but the experience of Lawrence and of Poughkeepsie would indicate that the chances of serious trouble from such growths are not very great. In this connection the writer desires to ask the author whether microscopical examinations of the water in the reservoir have shown as yet any tendency of the microscopic organisms to develop; whether the reservoir into which the filtered water is pumped has been recently cleaned; and, if not, whether there exists at the present time any considerable amount of sediment at the bottom of the reservoir. Experience with the reservoirs of the Brooklyn water supply, where mixed ground-water and surface water is stored in open sunlight, has shown that the deposits which form at the bottom of the reservoir have an important influence on the development of the microscopic organisms.

Allusion has been made to the growth of algæ upon the surface of open filters and to the annoyance which they cause in the operation of the filter. Some interesting studies of the growths of microscopic organisms over the sand have been made recently by Dr. Otto Strohmeyer, of Hamburg, and Dr. Ad. Kemna, of Antwerp. A brief account of their observations, with some additional studies made by the writer, may not be out of place in this discussion.

At Hamburg it has been the custom for a long time, whenever a filter-bed was scraped, to examine microscopically the surface film over the sand and to record the organisms present. Microscopical examinations of the water of the Elbe River have also been made. Much attention has been given to the careful enumeration of the different species, and, for the most part, the methods of the planktologists have been followed. The observations have shown that there is a regular seasonal succession of organisms which develop on the sand. During the winter the diatoms alone are represented, but certain species sometimes develop in great abundance. It is during the spring and fall, however, that the diatoms attain their maximum growth. The green algæ appear in the spring and increase during the summer. The blue-green algæ are present in large numbers during the late summer, their growth usually continuing until cold weather. Substantially, the same seasonal distribution was observed at Antwerp. It is interesting to note that it corresponds with the seasonal distribution of the microscopic organisms repeatedly observed in various lakes and reservoirs of this country.

At Antwerp similar studies of the surface scums have been systematically made, but somewhat different methods have been followed. The work has been carried on at the Waelham laboratory of the Antwerp Water-Works Company. Chief attention has been given to the dominant forms. The actual numbers of organisms present have not been

recorded, but the results have been expressed in proportionate parts of the total number present, on a scale of ten, as follows: Mr. Whipple.

Coscinodiscus	4½
Melosira	4
Cyclotella	1½
	10

On this date (Oct. 7th), therefore, the predominant forms were *Coscinodiscus* and *Melosira*. At another time the following genera were present:

Melosira varians	5
Fragilaria capucina	4
Spirogyra	1
	10

In this country similar results have sometimes been expressed in "number of organisms on 1 sq. cm. of sand." The following represents the results of a microscopical examination of the film at the top of an experimental sand filter. The sample was collected in March, after the filter had been in operation nearly two months:

	Number of organisms over 1 sq. cm. of sand. (In standard units.*)
Diatomaceæ:	
Asterionella	278 000
Cymbella	130 000
Diatoma	150 000
Melosira	10 000
Meridion	25 000
Navicula	7 700
Stephanodiscus	6 500
Synedra	1 100 000
Tabellaria	2 390 000
Chlorophyceæ:	
Closterium	1 200
Scenedesmus	800
Protococcus	60 500
Conferva	12 000
Spirogyra	5 500
Cyanophyceæ:	
Chroöcoccus	5 300
Oscillaria	84 000
Protozoa:	
Trachelomonas	16 000
Ciliata	5 000
Peridinium	4 000
Tintinnus	14 000
Mallomonas	800
Synura	6 000
Codonella	400
Rotifera:	
Anuræa	800
Polyarthra	1 000
Synchæta	8 000
Total organisms	4 324 500
Amorphous matter	2 300 000

* One standard unit equals 400 square microns.

Mr. Whipple. The organisms which develop over the surface of a sand filter may be grouped, for practical purposes, into three classes: those which form a matting upon the sand; those which are attached to the sand but extend upward in filaments or sheets; and those which are free-floating in the water. Perhaps it would be better to say that the organisms are found in these three conditions, because the same organism is sometimes found now on the sand and now above it.

The effects of these three groups of organisms upon the operation of the filter are not the same. The most important effect is that produced by those organisms which form a matting upon the sand. The diatoms and the unicellular algæ are here chiefly concerned. By their growth they form a more or less gelatinous film upon the surface, and as this film becomes denser, the rate of filtration is retarded until finally it becomes necessary to scrape the filter. The algæ which grow erect upon the sand do not thus clog the filter. On the contrary, they prevent clogging to some extent. Their waving, interlaced threads act as a sort of preliminary strainer, removing from the applied water some of the suspended matter which would otherwise collect on the sand. This action continues as long as the plants are in good condition and as long as the evolution of gas is sufficient to cause flotation. When they begin to decay or when they become overloaded with foreign matter they settle to the bottom and help to clog the filter. Kemna found that at Antwerp *Hydrodictyon* was the most effective organism in this process of preliminary straining. The free-floating forms have little influence on the rate of filtration as long as they remain in suspension, although, to some extent, they too play a part in the preliminary clarifying process. But ultimately most of them reach the surface of the sand and help to clog the filter.

Not only do the algæ growths over a sand filter affect the rate of filtration and the frequency of scraping, thereby increasing the cost of filtration; but they exercise an important influence upon the efficiency of the filter. It sometimes happens that the growth of the algæ is so vigorous and the evolution of gas so abundant that great masses of the organisms rise in the water, carrying with them patches of the surface film and leaving bald spots on the sand through which the water passes at too high a rate, with consequent loss of bacterial efficiency. At Antwerp the filter attendants watch for this phenomenon, and reduce the rate of filtration if necessary. It seems probable, also, that decomposition of the organisms at the surface affects the filtered water unfavorably.

During the course of the year the character of the flora changes. This change is often gradual, but at times is very rapid. Kemna has noticed that at the time when certain organisms are rapidly disappearing from the sand the efficiency of filtration is impaired. He attributes this to the changed condition of the surface film caused by

the decomposition of the organisms, but suggests that changes in the bacterial flora may also play an important part. In a recent publication* Dr. Ad. Kemna cites the following interesting experience with *Anabœna*: Mr. Whipple.

During the hot weather of July, 1899, *Anabœna* became abundant over some of the Antwerp filter beds. Knowing the character of this organism and its tendency to impart an odor to the water, he kept a careful watch of the filters, collecting samples of the filtered water twice a day and testing them as to their odor and the amount of ammonia they contained. As long as the *Anabœna* remained in a living condition in the water over the sand, the filtered water was satisfactory, but when the organisms disappeared, on the advent of cold weather, the filtered water acquired a bad taste and the amount of ammonia increased.

The studies made at Hamburg and at Antwerp show, with apparent conclusiveness, that when the vegetation over a sand filter is in a living condition, it is a positive aid to the efficiency of filtration, though it increases the cost of operation. Most of the microscopic organisms have a coating which is somewhat gelatinous, and in many cases the gelatinous material is very abundant. The diatoms and other organisms which grow directly on the sand aid in the formation of the surface film on which the efficiency of filtration largely, but not solely, depends. This fact has been understood for many years. The surface film forms through bacterial agency on covered filters as well as on open filters, but on the latter its formation is assisted by the microscopic organisms.

TABLE No. 8.—The Influence of Growing *Enteromorpha Intestinalis* upon Water Bacteria. (Direct Sunlight.)

Date.	Hour.	Temperature.	Number of Bacteria per Cubic Centimeter.	
			Culture of *Enteromorpha.*	Water without *Enteromorpha.*
July 6th	9 P. M.	18° Cent.	123	122
" 7th	5 A. M.	18° "	156	164
" 7th	9 A. M.	18.5° "	98	210
" 7th	3 P. M.	22° "	5	532
" 7th	7 P. M.	24° "	0	987
" 7th	9 P. M.	21° "	0	1 230

The experiments of Strohmeyer, at the laboratory of the Hamburg Water-Works, have shown that some algæ exercise a sterilizing influence upon the water in which they develop. *Enteromorpha*, for example, is a green alga which is often very abundant on the filter

*"Les Eaux Potables, Extrait de la Belgique Médicale."

Mr. Whipple.

beds during the summer. Strohmeyer selected and carefully cleaned some of the young, growing filaments of this organism and put them in flasks which contained about 300 c. c. of water, adding also 1.5 c. c. of a sterile culture solution. Other flasks of water were similarly prepared, but without the algæ. Pairs of these flasks, with and without the algæ, were subjected to similar conditions of light and temperature, and their bacterial contents determined at intervals of a few hours for several days. The result of one of these experiments is given in Table No. 8.

The figures show that at the beginning of the experiment the bacterial contents of the two flasks were practically the same. After standing for eight hours in the dark, the bacteria increased slightly in both flasks. After fourteen hours' exposure to direct sunlight, the water in the flask which contained the growing algæ was practically sterile, while in the other flask the development of the bacteria continued uninterruptedly. A similar experiment, carried on for a longer period in diffused light, gave the results shown in Table No. 9.

TABLE No. 9.—The Effect of Growing *Enteromorpha Intestinalis* upon Water Bacteria. (Diffused Light.)

Date.	Hour.	Temperature.	Number of Bacteria per Cubic Centimeter.	
			Culture of *Enteromorpha*.	Water without *Enteromorpha*.
July 4th	11.30 A. M.	18° Cent.	145	108
" 4th	2 P. M.	18° "	160	144
" 4th	6 P. M.	18° "	152	243
" 5th	8.30 A. M.	17° "	1 100	5 900
" 5th	2 P. M.	18° "	180	26 000
" 5th	6.30 P. M.	18° "	7	50 000
" 6th	9 A. M.	18° "	24	63 000
" 6th	7.30 P. M.	18.5° "	0	80 000

Here, the development of bacteria in the flask without the algæ was more vigorous than before, but the sterilizing action was less vigorous, as might be expected. Other experiments with *Spirogyra*, *Cladophora* and *Stichococcus* gave similar results. Whether the sterilizing action produced by the growth of the algæ was due to the effect of the gases liberated or to some other cause was not definitely determined.

What is true of these filamentous algæ is probably true of many other microscopic organisms, and, not only of the green algæ, but of the diatoms and, perhaps, the blue-green algæ. At the time when *Anabæna* was present on the Antwerp filters, Kemna observed that it was not equally abundant on all the beds, and that with the beds

where it was most abundant there was not only less clogging of the sand, but an increased degree of purification. Mr. Whipple.

The writer has had frequent occasion to observe the reduction of the number of bacteria in water by growths of microscopic organisms. The following case is very striking: A pond, with an area of 40 acres and a capacity of 42 000 000 galls., was affected with an immense growth of *Clathrocystis* which appeared in the spring and continued until the late autumn. At times the water contained 20 000 standard units of these organisms per cubic centimeter. The water that entered the pond was polluted, and the number of bacteria in it was seldom lower than 1 000 per cubic centimeter, and was often much higher. At the same time the number of bacteria in the water at the outlet of the pond was very low. On one occasion the following results were obtained, and these are typical of the conditions which prevailed for several months:

	Number of Bacteria per cubic centimeter.
Average of the influent streams	1 400
Average of samples throughout the pond	27
Sample at the outlet	36

A series of examinations of samples collected at the outlet of the pond during an entire year gave the following results:

	Number of bacteria per cubic centimeter.	Number of standard Units of *Clathrocystis* per cubic. centimeter.
January	.....	120
February	1 100	800
March	1 500	0
April	338	680
May	370	2 320
June	139	14 500
July	60	17 200
August	104	11 700
September	245	8 750
October	540	8 970
November	840	1 150
December	1 243	650

These figures show that the number of bacteria at the outlet of the pond varied inversely with the amount of *Clathrocystis* present. In this case a part of the reduction of the bacteria, at least, appeared to be due to a mechanical action by which the bacteria were engulfed in the mass of jelly in which the cells of *Clathrocystis* were embedded. Microscopical examination of the *Clathrocystis* showed that various kinds of bacteria were present in this gelatinous mass—micrococci,

Mr. Whipple. long bacilli, short bacilli, singly and in groups. The bacteria thus absorbed may or may not have been in a living condition, but at least they were incapable of developing on the gelatin plate. Microscopical examination of the cultures in the Petri dishes showed that in very few instances was a mass of *Clathrocystis* the nucleus of a colony of bacteria. Laboratory experiments were equally suggestive of the action of *Clathrocystis* on bacteria. A water which contained 430 bacteria per cubic centimeter was mixed with an equal volume of a water very rich in *Clathrocystis*, which contained but 39 bacteria per cubic centimeter, and shaken vigorously. Plate cultures of the mixed waters resulted in the development of only 60 bacteria per cubic centimeter. Other experiments of a similar character corroborated these results.

In the reservoirs of the Brooklyn Water-Works, it has been observed repeatedly that when *Asterionella*, *Synedra* and other organisms have been present in great abundance, the numbers of bacteria have been unusually low. There is some reason to believe that the microscopic animal growths tend to reduce the number of bacteria in water by consuming them as food. The relation between the water bacteria and the lower forms of microscopic plants and animals is certainly an intimate one, and the subject is one which deserves careful study, as it is not only important in connection with filtration, but has a direct bearing upon the self-purification of streams and other natural phenomena.

Mr. Rafter. GEORGE W. RAFTER, M. Am. Soc. C. E. (by letter).—This paper presents, in an interesting manner, the detail of a continuous filtration plant constructed substantially on lines followed for many years in England, and on the Continent of Europe generally. The cost of the plant per unit area is lower than that of the European plants, which is attributed to the favorable conditions at Albany. Basing his view on perhaps casual study of filtration plants abroad, the writer is, however, disposed to say that the decrease in cost is partly due, at any rate, to somewhat less elaborate construction than is common there. The extensive use of concrete has also contributed to keep the cost within moderate limits.

The author has referred to studies of the flow of the Upper Hudson, made by the writer. The cited figure of 1 647 cu. ft. per second, minimum flow, is probably not far from right, although in the summer of 1889, for a few days, the flow may have been as low as 1 200 to 1 300 cu. ft. per second.

Table No. 10 gives the run-off of the Hudson River in cubic feet per square mile per second, for a period of 12 years, from 1888 to 1899, inclusive. These figures do not represent the natural flow of the spring months, which are modified by a lumberman's storage of from 3 000 000 000 to 4 000 000 000 cu. ft. The Indian Lake Reservoir, of

4 470 000 000 cu. ft. content, was also brought into use in 1899, the storage therefrom being discharged into the stream during July, August and September. Had it not been for this large artificial inflow, the minimum of these months would have been considerably lower than shown. Mr. Rafter.

TABLE No. 10.—RUN-OFF OF HUDSON RIVER AT MECHANICSVILLE, FOR THE WATER YEARS, 1888 TO 1899, INCLUSIVE, IN CUBIC FEET PER SECOND PER SQUARE MILE OF CATCHMENT AREA.

(Catchment Area = 4 500 Square Miles).

Period.		1888	1889	1890	1891	1892	1893	1894	1895	1896	1897	1898	1899
Storage	December	1.78	2.22	2.93	0.72	1.91	0.90	1.60	0.97	2.42	1.54	3.05	1.22
	January	1.41	2.44	2.50	1.84	4.19	0.71	1.50	0.86	1.51	0.89	1.72	1.49
	February	0.82	0.84	1.76	2.58	2.06	1.07	1.08	0.79	1.04	0.87	1.50	1.17
	March	1.52	1.84	2.47	3.94	2.43	1.83	3.28	0.93	3.02	2.49	4.49	2.14
	April	4.73	3.04	3.34	4.44	4.79	3.98	2.47	5.31	5.55	4.24	3.07	5.25
	May	4.76	1.97	4.00	1.22	4.36	4.95	1.68	1.52	1.02	2.70	2.47	2.17
	Means	2.55	2.08	2.85	2.45	3.30	2.24	1.96	1.72	2.42	2.12	2.72	2.23
Growing	June	1.09	1.52	1.64	0.71	2.76	1.07	1.58	0.63	1.05	2.64	1.17	0.58
	July	0.34	1.28	0.43	0.52	2.08	0.56	0.70	0.50	0.62	2.39	0.57	0.54
	August	0.38	0.95	0.45	0.59	1.22	1.11	0.55	0.87	0.54	1.83	1.13	0.31
	Means	0.60	1.21	0.83	0.65	2.00	0.91	0.94	0.67	0.74	2.29	0.96	0.47
Replenishing	September	0.63	0.44	1.96	0.45	0.99	1.53	0.41	0.58	0.51	0.51	0.84	0.46
	October	1.02	0.88	2.04	0.33	0.63	0.86	0.81	0.58	0.91	0.56	1.75	0.58
	November	2.36	1.77	2.03	0.91	1.69	0.81	1.42	1.87	2.97	2.21	2.05	1.42
	Means	1.34	1.00	2.02	0.56	1.09	1.06	0.88	1.01	1.46	1.09	1.55	0.82
Water Year	Means	1.74	1.60	2.13	1.53	2.42	1.62	1.43	1.28	1.74	1.93	1.98	1.44

In his statements relating to cracks in the walls, the author apparently assumes that cracks in masonry are, on the whole, to be expected. This has always seemed to the writer so far an erroneous view that he is disposed to say that even in our severe winter climates, cracks in concrete masonry ought not to occur. At any rate, this general statement may be made for walls of any considerable thickness. For thin walls, the statement may possibly be modified somewhat, although there are certainly cases of such, of considerable length, free of cracks.

The writer is disposed to assign lack of cleanliness as a prolific source of so-called temperature cracks. As a general statement, for walls as long as those referred to by the author, the elasticity of the mortar ought to take care of expansion and contraction. But this implies thorough adhesion of the mortar. The chief remedy is, of course, cleanliness of stone and mortar materials.

By way of showing the elasticity of concrete under compression, reference may be made to the writer's studies of concrete as recorded in the Annual Report of the State Engineer and Surveyor of New

Mr. Rafter. York, for 1897, where tabulations of a large number of compression tests may be found. Without going into an extensive *résumé* of these tests, at this time, a few points may be cited in illustration of the writer's proposition.

Table No. 11 gives the serial number as per report to State Engineer, brand of cement, resilience and modulus of elasticity for a number of concrete blocks of 1 to 3 plastic mortar, 40% of the aggregate. The resilience here tabulated is for a gauged length of 5 ins., and a compression of 600 lbs. per square inch, while the modulus of elasticity is between loads of 100 and 600 lbs. per square inch.

TABLE No. 11.

Serial number.	Brand of cement.	Resilience for length of 5 ins.	Modulus of elasticity.
23	Genesee	0.0020	1 250 000
51	Wayland	0.0014	1 786 000
72	Iron Clad	0.0013	1 923 000
90	Empire	0.0011	2 273 000
109	Champion	0.0016	1 562 000
	Mean =	0.0015	.....

For a length of 1 ft., and with 600 lbs. per square inch compression, the mean resilience of the foregoing blocks is therefore found to be 0.0036 in. At this rate a concrete wall of the specified composition might be expected—if free to move—to expand nearly $1\frac{1}{2}$ ins. before serious consequences would result. Taking into account that so-called temperature cracks do not often exceed, even in walls several hundred feet in length, from $\frac{1}{8}$ to $\frac{1}{4}$ in., it has seemed probable to the writer that, with the proper precautions taken, they ought not to occur. At the same time, there is no intention of being specially insistent on these points, but rather to present them in the hope of eliciting further and, possibly, more profitable discussion.

For ordinary brick walls, the writer recognizes that they are very likely to crack under some of the conditions detailed in this paper.

Tables Nos. 11 to 16, inclusive, of the writer's report on Concrete, in the Annual Report of the State Engineer and Surveyor of New York, for 1897, give results of a large number of compression tests and computed moduli of elasticity for concrete and mortar. Additional data of this character may be found in the "Report on Tests of Metals and Other Materials, Watertown Arsenal, 1898." Indeed, recent compression tests have so multiplied as to enable one to form a fair idea of the elasticity of concrete and mortar. For other masonry materials there is still a great lack of data.

G. L. CHRISTIAN, Assoc. M. Am. Soc. C. E. (by letter).—This paper is a valuable addition to the literature of water filtration. The large death rate from typhoid fever before the construction of the filter, and the material reduction in the deaths from that cause alone would justify the expenditure necessary for its installation. But, when all the inhabitants use the filtered water it would seem that the death rate should be decreased even more than it has been. Mr. Christian.

The writer was interested in the cracks in the concrete and brick masonry caused by the low temperature. He was also interested in Mr. Rafter's remarks on that subject, having had a similar experience some years ago while engaged on a reservoir in which there was a straight masonry wall approximately 475 ft. long. The height of this wall from the bottom of the reservoir to the under side of the coping was 26 ft. Its thickness was 4 ft. at the top and 13 ft. 6 ins. at the bottom.

It was built of second-class rubble masonry, the constituents being a freshly quarried gneiss of good quality and a mortar composed of 2 parts sand to 1 of a standard American Portland cement.

The rock was all cleaned and washed immediately before being laid, and the work was done by good masons, of experience in that line of work, and who were under constant supervision. The wall was completed during the latter part of August, and, as cold weather approached, it was examined carefully every few days.

Early in December the thermometer fell suddenly to 12° Fahr., and on that day cracks were noticed, as shown in Fig. 18. They extended

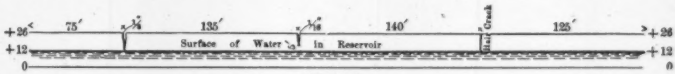


FIG. 18.

through the wall, were about at right angles to it, and generally followed the joints. They never widened, but, on the contrary, closed gradually as the water rose in the reservoir and the warm weather approached, finally closing so tightly that only by the closest scrutiny could they be seen, even though each place had been marked carefully in order that it might be found readily.

JOHN C. TRAUTWINE, Jr., Assoc. Am. Soc. C. E. (by letter).—To the writer this paper has a special interest as a source of information and a basis of comparison, in connection with the several large plants now being designed for the filtration of the Philadelphia supply. Mr. Trautwine.

After innumerable reports upon filtration, by Councilmanic Committees, by local organizations,* by the writer, as Chief of the Bureau of Water, and by his predecessor, Mr. John L. Ogden, and after years of fruitless discussion and inaction, the present Mayor, Hon. Samuel H. Ashbridge, who took his seat on April 3d, 1899, secured from

* Including the Woman's Health Protective Association, which submitted a report by Mr. Jos. B. Rider and, later, one by Mr. Allen Hazen.

Mr. Trautwine. Councils, on April 20th, an ordinance providing for "the employment of three experts relative to the improvement, filtration and extension of the water supply," who were "to act in conjunction with the Director of the Department of Public Works, Chief of the Bureau of Water and Chief of the Bureau of Surveys in examining and reporting upon the question."

On May 8th, the Mayor announced the appointment of Messrs. Rudolph Hering, of New York; Samuel M. Gray, of Providence, and Joseph M. Wilson, of Philadelphia, Members, Am. Soc. C. E., who, on September 15th, presented a report recommending, in conclusion:

1. The adoption of that project by which the waters of the Schuylkill and Delaware Rivers, taken within the city limits, are purified by filtration.

2. The immediate improvement of the existing plant, in accordance with the detailed recommendations of their report.

Among the improvements recommended was the restriction of waste by the introduction of meters.

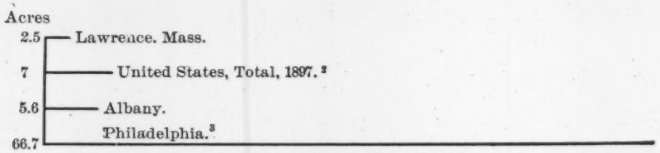
In their *résumé* and conclusions the experts say:

"We consider that, at present, a daily supply of 200 000 000 galls., being 150 galls. per capita, is a very liberal allowance. We recommend that this quantity of pure water be immediately provided for."

This involves a material restriction of the consumption, which is now variously estimated at from 220 000 000 to over 275 000 000 galls. daily.[1]

The striking features of the Albany plant are, perhaps, on the one hand, its immensity when compared with previously existing American slow filtration plants, and on the other hand, its insignificance (if the word may be used without offence), in comparison with the magnificent installation recommended by the experts for Philadelphia.

Prior to the construction of the Albany plant, with its aggregate filter-bed area of 5.6 acres, that at Lawrence, Mass., with a corresponding area of 2.5 acres was much the largest slow plant in America, and in 1897 the aggregate area of all those plants of which the writer could learn did not reach 7 acres (Fig. 19).[2]



COMPARISON OF AREAS OF FILTER BEDS.

FIG. 19.

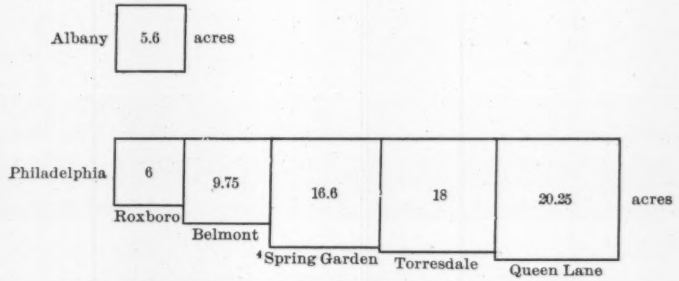
[1] Two hundred and twenty million U. S. galls., as estimated by the experts. By plunger displacement, after allowance for slip, the pumpage, as stated in Report of Water Bureau for 1898, was 275 000 000 galls.

[2] Mr. Hazen informs the writer that the aggregate area in the United States, in 1897, as then calculated by him, was a little over 9 acres. See *Journal* of the New England Water-Works Association, Vol. xii, p. 229.

[3] For a supply reduced to 200 million gallons per day, as recommended by the experts, at 3 million gallons per acre per day.

Upon visiting the Albany plant during its construction, in May, 1899, the writer was much impressed with the great extent of the operations in progress, indicated by the photographic views accompanying Mr. Hazen's paper, and it was therefore a startling reflection that these vast provisions would be insufficient to supply one of the four 20 000 000-gall. pumps at the Queen Lane (Philadelphia) pumping station, which station represents barely one-fifth of the aggregate nominal capacity of all the Philadelphia pumps, and that the daily pumpage at Philadelphia[1] was nearly fifteen times greater than the capacity of the Albany plant. Mr. Trautwine.

Even with the restricted consumption contemplated by the experts, the Philadelphia filtration plant will be nearly, if not quite, the largest in the world, and certainly very much larger than any other under a single control.



COMPARISON OF AREAS OF FILTER BEDS, ALBANY AND PHILADELPHIA.
FIG. 20.

Philadelphia is supplied from six stations, the smallest of which (Roxborough) has a nominal daily capacity of 24 500 000 galls., and is credited, in the Bureau report for 1898, with an average daily pumpage of over 20 000 000 galls., as against 15 000 000 galls. nominal daily capacity, and 12 500 000 galls. actual consumption, for the Albany plant.

The aggregate bed area of the four slow plants recommended by the experts for immediate construction for Philadelphia is 54 acres, and, in addition to this, a rapid or "mechanical" plant of 50 000 000 galls. daily capacity was recommended for the Spring Garden System.

The filter-bed areas of these four plants, and the equivalent slow-bed area of the rapid plant recommended for the Spring Garden System[5] are shown graphically, and compared with that of the Albany plant, in Fig. 20.

[1] Two hundred and twenty million U. S. galls., as estimated by the experts. By plunger displacement, after allowance for slip, the pumpage, as stated in Report of Water Bureau for 1898, was 275 . 00 000 galls.

[4] "Rapid" plant, 50 000 000 galls. daily, taken as equivalent to $\frac{50}{3}$ = 16.6 acres of slow beds.

[5] Capacity allotted, 50 000 000 galls. per day. Rate assumed, 3 000 000 galls. per acre per day.

Mr. Trautwine. The construction of the Albany plant happens most opportunely for the authorities in charge of the Philadelphia work, who may profit by it, not only as a pattern, but, if time permits, as a most useful monitor, showing where modifications may be advisable in the Philadelphia plants in order that they may be more perfectly adapted to the conditions there existing. Indeed, unless the differences in conditions between the two places are studied most carefully, the Albany plant will lose much of its special usefulness to the Philadelphia authorities.

Philadelphia draws more than 95% of its present enormous supply from five pumping stations on the Schuylkill River, whose water-shed is about 1 900 square miles, and the remainder, or less than 5%, from the Delaware, the water-shed of which, above Philadelphia, is between four and five times as large as that of the Schuylkill.

Between the Hudson, at Albany, and the Delaware, at Philadelphia, there is considerable analogy. Both cities draw from tide-water; each is within a half day of the sea, by water; in each case city sewers discharged into the river both above and below the intake prior to filtration; in each the design of the filtration plant involves the removal of the intake to a point above the discharge of city sewage; and the water-shed areas of the two rivers, above the two cities, respectively, are nearly equal.

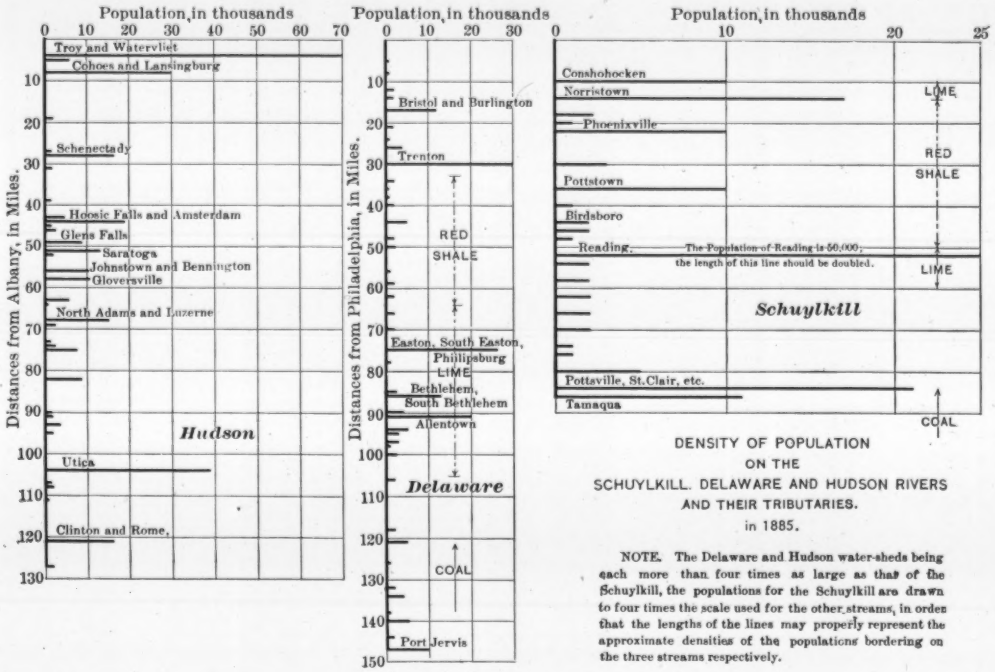
Mr. Hazen gives the population per square mile of the Hudson, above Albany, as 33 in 1880, and 43 in 1890, from which we may assume 38 as the corresponding figure for 1885;* while Mr. Hering, in his report on extension of water supply in 1885, gives the population of the water-shed of the Delaware, above Philadelphia, as 59 per square mile, including the Lehigh water-shed, and 54 exclusive of that water-shed.

The distribution of the population adjacent to the river, in each of the two water-sheds, is shown graphically in Fig. 21, in which they are compared also with that of the Schuylkill.

All the data of population shown in Fig. 21, are those for the year 1885, they having been found convenient of access in each case. While those of to-day would of course show a marked increase in most cases, yet the changes which have occurred are probably not of such a nature as to interfere with the usefulness of the figures given, for the immediate purpose in hand, viz., an exhibit of the relation between the three rivers in regard to density and distribution of population.

For the Schuylkill and Delaware the populations have been taken

*In his letter closing the discussion Mr. Hazen reminds the writer that these figures, for the Hudson embrace only the urban population, and states that the rural population is nearly constant at about 30 per square mile. This would make the total population, for the Hudson water-shed, in 1885, 68 per square mile, or about 15% greater than that of the entire Delaware water-shed, including that of the Lehigh, and 26% greater than that of the Delaware, exclusive of the Lehigh.



DENSITY OF POPULATION
ON THE
SCHUYLKILL, DELAWARE AND HUDSON RIVERS
AND THEIR TRIBUTARIES.
in 1885.

NOTE. The Delaware and Hudson water-sheds being each more than four times as large as that of the Schuylkill, the populations for the Schuylkill are drawn to four times the scale used for the other streams, in order that the lengths of the lines may properly represent the approximate densities of the populations bordering on the three streams respectively.

FIG. 21.

Mr. Trautwine. from a map prepared by Mr. Rudolph Hering, and published with his report of 1885 on the improvement of the Philadelphia supply. In the case of the Hudson they were found by averaging the figures given by Mr. Hazen, in Table No. 1, for the years 1880 and 1890. The distances, for the Schuylkill and Delaware, were roughly scaled from the map mentioned. Those for the Hudson are given by Mr. Hazen in Table No. 1. In no case has the writer made any attempt at accuracy.

A striking feature, in the case of the Hudson, is the large population massed upon the banks of the stream at Troy and Watervliet, only 4 miles above Albany, with a further addition of population, half as great, within the next 4 miles, giving a total population (in 1885) of 100 000 within 8 miles of the intake. Nothing like this exists on the Delaware. The portion of Philadelphia lying above the intake has, in general, only a rural population, and the experts have recommended the removal of the intake to a point just within the upper city limit. The largest town on the Delaware (Trenton, N. J.) had, in 1885, a population of only about 30 000, or less than half of that of Troy and Watervliet, and it is 30 miles above the city. The entire population represented on the diagram as contiguous to the Delaware, within 50 miles of Philadelphia, does not equal that on the Hudson within 5 miles of Albany. Nevertheless, the density of population on the entire water-shed of the Delaware, as given by Mr. Hering, for 1885, is more than 50% greater than that for the Hudson, as deduced from Table No. 1.*

But while the conditions on the Hudson and the Delaware are thus seen to be somewhat analogous, a glaring contrast appears when we compare either of these two rivers with the Schuylkill; and, inasmuch as Philadelphia now takes more than 95% of her supply from the latter stream, and is to continue to take 75% from it, according to the plan recommended by the experts for immediate execution, the importance of bearing these differences in mind becomes at once apparent.

The water-sheds of the Delaware and the Hudson are each between four and five times as large as that of the Schuylkill, and the density of population is approximately in inverse proportion to the areas. On the map accompanying his report of 1885,† Mr. Hering states the average population of the Schuylkill water-shed above Philadelphia as 176 per square mile, while Mr. Dana C. Barber, in a table accompanying his report of 1884 upon his sanitary survey of the Schuylkill Valley,‡ gives the area as 1863.9 square miles, and the population as 372 000, making the average density of population 200 per square mile. In order to represent with approximate correctness the rela-

* See foot note * page 330.

† Report of Water Department, Philadelphia, for 1885.

‡ Report of Water Department, Philadelphia, for 1884.

tions between the three rivers in the matter of density of population, the writer has plotted the populations for the Schuylkill upon a scale four times as large as for the other two rivers. Mr. Trautwine.

Referring to the diagram of the Schuylkill, we find, within about 50 miles of the city, Conshohocken, Phœnixville, Pottstown and Birdsboro, all important iron manufacturing towns, and Norristown, a well-to-do county-seat, once important in the same respect, while, near the head-waters, are Pottsville and Tamaqua, both important anthracite coal mining centers.

Most of these towns are without sewerage systems. Reading is installing such a system, but, with commendable consideration for its neighbors down stream and in the absence of pressure from without, is also installing plants for the filtration of all the sewage led to the river.

Within the city limits of Philadelphia (and, therefore, not shown) and above all but the Roxborough station, is the important textile manufacturing suburb of Manayunk, with a population probably between 10 000 and 15 000. The sewage proper of Manayunk, including most of the discharges from the mills, all of which, until recently, went into the river, is now carried through an intercepting sewer, completed in 1888, to a point below the Fairmount dam, which separates the entire pumpage system from tide water, but much household and other filth is carried directly into the river by storm water, which is not admitted to the intercepting sewer; and probably much more (including, according to official reports, fæcal matter from dwellings of foreign laborers adjacent to the canal of the Schuylkill Navigation Company) is thrown into the canal, from which it passes into the river, barely a mile above the Queen Lane pumping station, the newest and finest of the city's water-works.

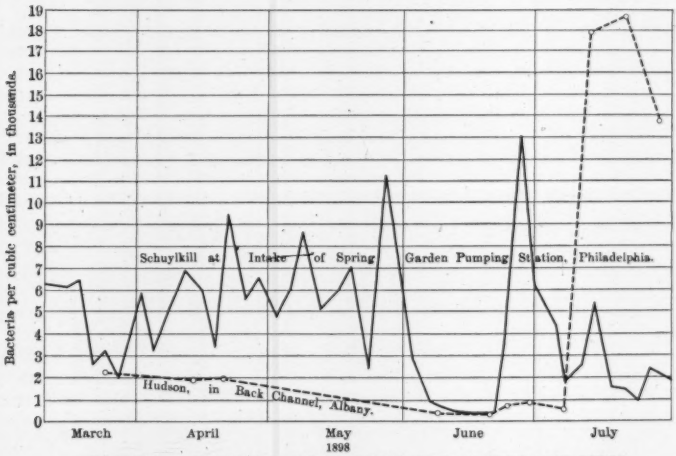
The canals of the Navigation Company, passing, as they do, through most of the towns along the river, and furnishing power to mills there, are naturally made the receptacles of offal, and the nuisance is especially flagrant beyond the city limits, where the city's Board of Health has no jurisdiction. A trip through the canal opposite Norristown, for instance, reveals great accumulations of filth of the most revolting description, scarcely awaiting the next rain to pass into the canal, and thence, in due course, into the river.

The Schuylkill, however, has this advantage over the Delaware and the Hudson, that the two pools, from which the pumps draw their supplies, are at least shut off from tide-water, and thus from the major part of the city's own sewage, by the dam at Fairmount.

Apart from sewage pollution, the Schuylkill suffers mineral pollutions of aggravated character.* In the anthracite regions, at and near

* See Fig. 21, in which the limits of the significant geological formations are indicated both for the Schuylkill and for the Delaware.

Mr. Trautwine. its source, it receives sulphuric acid, produced by the oxidation of the iron sulphide occurring in the coal, and large volumes of anthracite coal dust from the washeries, established during recent years for the purpose of extracting the small merchantable sizes from the culm or waste heaps which have been accumulating ever since the opening of the region, and from the "wet breakers," or breakers in which water is used for cleaning the coal and assorting it by sizes. The sulphuric acid is completely neutralized by the limestone of two extensive beds, one just above Reading and the other just above Philadelphia, and the water reaches the city with a basic or "hard" reaction.* The coal dust accumulates in the pools of the Navigation Company, but is swept out



BACTERIA IN SCHUYLKILL AND IN HUDSON RIVER WATERS, UNFILTERED.

FIG. 22.

and brought down to the city two or three times a year or oftener by floods.

Again, between Reading and Norristown, the Schuylkill passes a broad belt of easily decomposable red shale. In times of flood, this formation sends vast volumes of its substance, in suspension, into the stream, which, at such times, runs blood-red. The first effect of a general storm is, therefore, a visitation of red mud, which, a day or two later, is followed by water charged with coal dust. The writer has seen water drawn from faucets in Philadelphia, after storage in the city reservoirs, so black from coal dust as to be scarcely distinguishable from ink.

The Delaware (see Fig. 21) passes through practically the same

* The Delaware water, on the contrary, is quite soft, forming no scale in boilers.

formations, and receives its modicum of the same adulterants, but, owing to its much larger volume of flow, their effect upon the character of its water is much less marked. Mr. Trautwine.

Under the circumstances stated, it is not surprising that Fig. 22 shows a very much higher average number of bacteria in the Schuylkill water than in that of the Hudson (taken from Table No. 2), until after the "abnormal" conditions following July 9th set in, when the contractors on the Albany work "dumped sand and gravel in the back channel, and took it up again by dredging, for construction purposes, with the result that this water was fouled, and the samples taken after that time do not represent its normal condition."

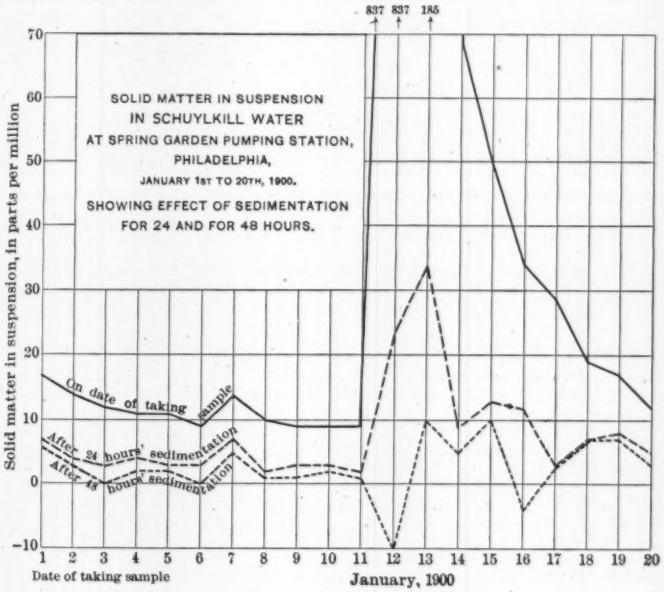


FIG. 23. (See page 336.)

Unfortunately, the writer is not in position to make a similar comparison between the two rivers in the matter of turbidity. During 1898 (the year for which Mr. Hazen's data in Table No. 2 are given), arrangements had not been completed for using the Hazen scale in observing turbidity in the Schuylkill, and the writer has not succeeded in deriving, from the examinations thus far recorded, a satisfactory coefficient for deducing the readings of the Hazen scale from the record, in parts per million, in which the Philadelphia results for

Mr. Trautwine.

1898 are stated. The Philadelphia results for March–July, 1899 (measured by the Hazen scale), do not differ greatly from the Hudson results as given by Mr. Hazen; but Mr. George I. Bailey, in his discussion, states that the highest turbidity reached since the filters were put in operation was 0.60; whereas, during the same period, it has on three occasions reached or exceeded that figure in the Schuylkill, as follows:

August 11th–12th, 1899	0.80
September 27th–29th, 1899	1.50 to 0.60
January 12th, 1900	0.90 to 1.50

In the course of the investigations of the experts, during the summer of 1899, sets of three samples each were taken daily from the Schuylkill and from the Delaware, and examined for (1) the total dry residue contained, (2) the amount of such residue deposited during the first 24 hours, and (3) the amount deposited during the first 48 hours. These observations are still being made, and the Schuylkill results for January 1st–20th, 1900, are indicated in Fig. 23, from which appears the startling result that in some cases the amount deposited exceeded the amount originally contained in the water. In others (not shown), the results indicated that less sediment was deposited in 48 than in 24 hours. These erratic results are due, no doubt, to differences in the samples, although these were taken at approximately one and the same hour and place.

Notwithstanding that these investigations indicate in many cases a very nearly complete deposition of all the sediment during the first 24, or at furthest during the first 48, hours, the water, as a matter of fact, often remains visibly turbid for many days thereafter.

Fig. 24 shows a comparison between the Schuylkill and the Delaware as to total solids in suspension during three periods of flood, from which it appears, as might have been expected, that in the smaller stream this effect of floods is not only much more marked and generally of shorter duration, but also appears and disappears earlier than in the larger stream. During normal stages, the sediment in both rivers ranges ordinarily between 10 and 30 parts per million.

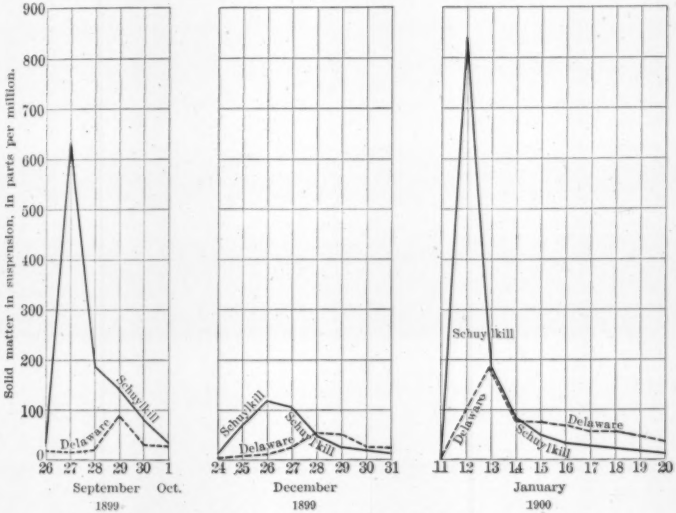
These comparisons show that the Schuylkill filters at Philadelphia will not only have a heavier bacterial duty to perform than those at Albany, owing to the greater number of bacteria, but will also be handicapped by the heavier doses of sediment in its water.

So far as the writer knows, there are on record no data which enable us to judge of the effect, upon the efficiency of the filters, of the coal dust brought down the Schuylkill and Delaware Rivers at times from the anthracite coal regions.

In the writer's judgment, the conditions stated render advisable a high degree of caution in using the construction and operation of the

Albany plant as a precedent in the design of the vastly larger system required for the filtration of the quantities of water taken daily from the Schuylkill for the supply of Philadelphia. In taking leave of the city service in November last, the writer addressed to the Mayor a communication, urging (as in many previous communications) that one or two of the smaller plants be built first, and "that the construction of filtration plants for the rest of our supply shall proceed as rapidly as we acquire, from the two initial plants to be immediately constructed, the knowledge so essential for such an undertaking." It is therefore gratifying to find his Honor quoted as saying recently that, after the

Mr. Trautwine.



SOLID MATTER IN SUSPENSION IN SCHUYLKILL AND DELAWARE WATERS AT PHILADELPHIA, IN TIMES OF FLOOD.

FIG. 24.

plans are completed, which will be "when thorough tests have been made at the new testing station," "it will then be in order to invite proposals for the work of constructing filtration plants at one or more of the city's stations."

With regard to that large portion of its supply which is drawn from the Schuylkill, Philadelphia is fortunate in having at her disposal the large reservoirs heretofore used for storage of the water after pumping, and incidentally for a very imperfect improvement in quality by means of sedimentation. In a report dated February 16th, 1898, to the Director of the Department of Public Works, the writer suggested that the reservoirs be utilized as sedimentation basins, a small portion

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of their total capacity being set apart for the storage of filtered water, and this feature was an essential one in a system of filtration suggested by the writer on September 9th, 1898, as well as in that recommended by the experts in their report of September 15th, 1899.

As regards the small portion of its supply taken from the Delaware, Philadelphia, like Albany, is without reservoir capacity of a kind which could be utilized conveniently for sedimentation of the water as a preliminary to filtration. Of the two small reservoirs of that supply, the one which has sufficient elevation is small and defective, and has but one basin. Both the experts' and the writer's proposed systems therefore provide for the construction of sedimentation basins for the Delaware supply.

An attempt to compare the cost of construction of the Albany plant, as given by Mr. Hazen in Table No. 3 and in his accompanying remarks, with that of the four proposed slow beds for Philadelphia, as estimated by the experts, is complicated by the difficulty of placing the two systems, with any certainty, upon one and the same basis of comparison. Mr. Hazen says, regarding the Albany plant:

"The filters, sedimentation basin and pure-water reservoir are connected in such a way as to make an exact separation of their costs impossible; but, approximately, the sedimentation basin cost $60 000, the pure-water reservoir $9 000, and the filters $255 000."

This makes the cost of the sedimentation basin about 18½% of the whole.

TABLE No. 12.—Cost of Filter Beds and Appurtenances at Albany.

(See page 340.)

(*a*) Items common to filters, pure-water reservoir and sedimentation basin.		(*b*) Items proper to filter beds and pure-water reservoir.	
		Brought forward,	$125 254.59
Preliminary draining	$1 956.71	Gravel for lining	1 508.75
Excavation	21 761.64	Stone for lining	1 850.74
Embankment	8 340.80	Concrete in vaulting	29 999 20
Filling	3 360.00	Brick work	35 603.75
" rolled	3 960.00	Filter gravel	7 645.05
Puddle	8 973.25	Filter sand	36 100.00
Concrete in floors	27 112.47	Vitrified pipe	7 153.32
Other concrete	6 703.11	Manhole covers	2 956.80
Cement	61 368.52	Sand-run fixtures	3 260.00
Extra work and minor items	10 150.11	Regulator houses	6 897.92
		Fence	1 704.00
	$153 686.61		$259 934.12
81½ per cent.	$125 254.59	Deduct cost of pure-water reservoir	9 000.00
			$250 934.12

TABLE No. 13.—Estimated Cost of Slow Filter Beds and Appurtenances, for Philadelphia. Deduced from Report of Experts. (See page 340.)

Plant.	Belmont.		Roxborough.		Queen Lane.		Torresdale.		Total.	
Capacity, in United States gallons per day.	27 000 000.		15 000 000.		58 000 000.		50 000 000.		150 000 000.	
Clearing	$9 000	.....	$8 000	.....	$10 000	.....	$10 000	.....	$37 000	.....
Excavation	31 200	.....	19 249	.....	66 382	.....	55 383	.....	172 214	.....
Sand washers*	3 200	.....	2 200	.....	7 200	.....	5 600	.....	18 200	.....
Drains	1 530	.....	1 130	.....	7 390	.....	6 060	.....	16 110	.....
Beds	490 568	.....	303 697	.....	1 009 949	.....	904 236	.....	2 708 450	.....
Fence	15 000	Add 15%†	10 800	Add 15%†	15 600	Add 15%†	5 000	Add 15%†	46 400	Add 15%†
Total	$550 498	$633 073	$345 076	$396 838	$1 116 521	$1 283 999	$986 279	$1 134 221	$2 998 374	$3 448 129
Per million gallons daily	20 389	23 447	23 005	26 456	19 250	22 138	19 725	22 684	19 989	22 987
Excess over Albany ($16 729).	22%	40%	38%	58%	15%	32%	18%	36%	19½%	37%

* Including "piping to sand washers." † 15% added by experts, presumably to cover "engineering and contingencies."

Mr Trautwine. Taking the items under "Filters, Sedimentation Basin and Pure-Water Reservoir," in Table No. 3, separating those supposed to be proper to the filter beds and the pure-water reservoir from those common also to the sedimentation basin, using only 81½% of the cost of the latter items, omitting items not included in the Philadelphia experts' estimates, as quoted in Table No. 13, and, finally, deducting $9 000 as the cost of the pure-water reservoir, we arrive at the estimate of the cost of the Albany filter beds, given in Table No. 12, or, taking the capacity at 15 000 000 galls. per day, $16 729 per million gallons of daily capacity.

For the four slow plants, at Philadelphia, we have the figures in Table No. 13.

The figures for Philadelphia, in Table No. 13, include excavation for a pure-water basin for Belmont and for Torresdale, and for a sedimentation basin at the latter station, but, from the contours of the sites it would appear that the major portion of the excavation, in each case, was for the filter beds proper. Besides, in the case of the Albany plant, no doubt much more than the 81½% which we have taken for excavation is chargeable to the sedimentation basin.

Much of the very considerable difference between the cost at Albany and the estimates of the experts for Philadelphia may, no doubt, be explained by the advance in prices between the execution of the Albany contracts and the making of the Philadelphia estimates; but, after making all allowances, it would appear that the experts' estimates, at least after their addition of 15%, are certainly safe.

Table No. 14 is a comparison of the total cost of improvement.

TABLE No. 14.—Comparison of the Total Cost of Improvement.

	Philadelphia (estimated).		Albany (actual).
	(*a*)	(*b*)	
Population assumed	1 333 333	1 333 333	95 000
Average daily consumption, in gallons per capita	150	75	158
Total gallons	200 000 000	100 000 000	15 000 000
Cost of installation:			
Land	$1 120 353	$100 000	$8 290
Filter plants, piping, sedimentation basins and accessories	6 818 468	3 235 000	323 960
Pumping stations	1 514 770	56 500	49 745
Mains	1 520 000	432 000	86 638
	$10 973 591	$3 823 500	$468 633
Waste restriction	100 000	1 000 000	0
Additions to and improvement of existing plant	3 290 500	1 107 000	0
Engineering and contingencies	Included.	Included.	28 000
Total	$14 364 091	$5 930 500	$496 633
Per million gallons daily	$71 820	$59 305	$33 109

Mr. Trautwine.

Of the two estimates given for Philadelphia, (*a*) is that of the experts, taken from pages 107, 122 and 123 of their report, and (*b*) is based upon a report by the writer, made September 9th, 1898, to the Director of the Department of Public Works, in response to a resolution of Councils. In order to make estimate (*b*) properly comparable with the other two, it has been amended by adding 50% to the estimated cost of the filters, to provide for their roofing, and an item of $700 000 to provide for roofing the clear-water reservoirs.

The resolution requesting this estimate asked for "plans and drawings and estimated cost of filtration of all the water used by the city," and the writer took advantage of the word "used," to show the city fathers what could be done, in the way of proper economy, by restricting the consumption to a figure (maximum 100 galls. per capita per day), more nearly commensurate with the quantity (possibly 50 galls. per day, maximum) really used.* The estimate was based upon the Albany bids, which then had just been opened, and the amounts of which were kindly communicated to the writer by Mr. Hazen for the purpose of the estimate.

The figures indicate an economy out of proportion to the proposed reduction of consumption, but this was to be expected. For the smaller quantity, sites were generally available on city property adjoining or near to existing plants, whereas, for the larger quantity, sites for the filter beds had, in all cases, to be acquired. In other ways, also, and notably in the matter of mains to connect existing works with filter beds perforce located at considerable distance, the total cost, in Philadelphia, increases much more rapidly than the quantity to be filtered.

Philadelphia lies just south of the line which Mr. Hazen has drawn to indicate at what locations it is advisable to roof over the filter beds, and the question of the advisability of doing so for Philadelphia may be one for experts to discuss. Poughkeepsie, after some twenty years' experience with an uncovered bed, built another, and the writer, largely on the strength of this, omitted roofing, in designing for Philadelphia the system already mentioned, but he has since been informed that the building of a second uncovered filter bed at Poughkeepsie was contrary to the advice of Mr. Fowler, the Superintendent.

As to cost of operation, the experts make estimate for Philadelphia, as follows:

"Per million gallons of filtered water, including labor, cost of wash and waste water, lost sand, sanitary analyses of water, chemicals, superintendence, watchmen, ordinary repairs, and all incidental expenses; but excluding interest, depreciation and cost of pumping water to filters:

	Schuylkill River.	Delaware River.
Slow filters.....	$3.60	$3.00
Rapid filters.....	4.80	4.00

* The result was a resolution, by the Water Committee, requesting the Department "to place in proper form the results of its researches on the question of Slow Sand Filtration for the entire city."

Mr Trautwine. "Cost of pumping, per million gallons raised 1 ft. high, including coal, labor, oil, waste and supplies, and ordinary repairs; but excluding interest and depreciation:

"Low-Lift Pumps.

"For a daily supply of 200 000 000 galls., 5.25 cents."

In his report of September 15th, 1898, the writer, with much misgiving, due to insufficiency of data, ventured an estimate of $3.97 per million gallons as the cost of slow filtration proper, and $0.41 per million gallons as the cost of raising the water by low-lift pumps, making a total of $4.38.

Considerable research, including a correspondence with all the American filtration plants of which the writer could learn, developed costs of filtration alone, ranging all the way from $1.50 to $10 per million gallons filtered, the latter figure represented by Lawrence, Mass., and the writer concluded that:

"In the absence of more exact information, and in view of the excessive turbidity of our water in flood and of the rate of wages fixed by ordinance of Councils for an 8-hour day's labor, it would be unsafe to estimate the cost of operation (exclusive of interest on cost), for the filters contemplated, at less than $3 per million gallons, notwithstanding the large dimensions of the proposed works, which should conduce to economy, and notwithstanding that the proposed reduction in consumption would greatly facilitate sedimentation. On the other hand, with proper economy, the cost should not exceed $5."

Mr. Bailey's experience with the Albany filters, as deduced from their operation from September 5th to December 25th, 1899, inclusive, and stated in his discussion, shows a cost, for filtration proper and laboratory work, of $1.67, and $2.52 for raising the water from the river to the sedimentation basin, making the total cost $4.19 per million gallons.

Mr. Soper. GEORGE A. SOPER, C. E. (by letter).—In the discussion of this valuable paper which has thus far been printed, attention seems to have been centered mainly upon the design and construction of the filters, a direction which was naturally suggested by the title of the paper itself. The fact that the question of purifying the Hudson River water for the City of Albany involves a great deal more than the mere design of filter-beds, with their piping and other fixtures, however, seems to warrant a more general survey of the matter.

At the outset, the problem, as stated by the author, was the purification of a water which was very heavily polluted with sewage and was believed to have contributed greatly to a high typhoid-fever death rate. The author says that, previous to his connection with the case, investigations relative to methods of purifying the supply had been made, and that the plan which he formulated, when the matter came before him, was based partly upon data which had been thus obtained. Briefly stated, it was recommended that a new intake be built, and that a settling basin and filters be constructed to treat the

water. Strictly speaking, the complete scheme included, also, a new pumping station, a system of aeration, a clear-water conduit and provision for storing the filtered water. Mr. Soper.

The storage of the filtered water has already taken up some space in the discussion, and the possibility that the clear water may develop organisms under the present circumstances, has been pointed out. In order to throw more light upon the subject, it is hoped that the author will supply information concerning the relative proportions of filtered and raw surface waters which will be stored together in the open reservoir, and the approximate length of time during which the mixture will remain exposed to the sun.

The purification plant was to consist of two parts: A settling basin and filters, and, as a means of obtaining a favorable quality of water, to begin with, the intake was to be placed in a side channel of the river. It is to be observed that the aim was to produce a water of unobjectionable quality, and that in the general scheme, two important measures, besides filtration, were to be taken to secure pure water. Filtration was to be used to complete bacterial removal, but the water was first to be obtained in as pure a state as possible, and then modified in a manner which would be favorable to that process. The weight and character of the suspended matter which is found in the Hudson River water, normally and after storms, has not been stated, and we are not informed as to the degree of modification which was considered necessary. Yet it would appear that these were matters of prime importance in influencing the choice of a design.

Keeping in mind the fact that not comparative purity, but a quality of water which would agree with high standards of bacterial purity, was demanded, it must be stated that the final product of the plant does not appear to be wholly satisfactory. The author has informed us, in the first appendix of his valuable book on filtration,* what it is reasonable to expect a filter to produce. According to the rules of the German Board of Health, the number of bacteria in the effluent of a filter treating surface water, should not exceed 100 per cubic centimeter. The data thus far supplied show that the Albany average exceeds this figure, and that for the first month the number of bacteria in the effluent was more than twice, and for the last month nearly twice, that required by the German standard. On occasions, which have been frequent enough to attract notice, the bacteria in the effluent have exceeded 500 per cubic centimeter.

It is a curious fact, but one based upon considerable observation, that the efficiency of a filter, as stated in terms of percentage of removal, is commonly understood to be a definite indication of the quality of the effluent. This is a mistake, since a 99% removal,

* "The Filtration of Public Water Supplies."

Mr. Soper. although showing that a large proportion of the impurities have been removed, gives no indication whatever as to the amount of pollution which remains. Yet, as has been pointed out, it is the pollution which remains in the effluent which is of importance. To quote an illustration, and to apply this reasoning to the present case, the Albany filters have shown for four months, according to the Superintendent's figures, a reduction in bacteria of 99%, while the average number of bacteria left in the water was 168 per cubic centimeter. This excludes the figures for the first week, which do not appear to represent a normal operation. An examination of the tables shows that there have usually been upwards of 200 bacteria in the effluent when the numbers in the raw water have exceeded about 13 000.

The importance which attaches to the foregoing remarks has a general application, and leads to a query as to the number of bacteria which we may consider it good practice to allow upon a filter of this type. Obviously, there should be a limit, if the effluent is to be narrowly controlled. If the quality of the final product of a purification plant is to be fixed at a standard of, say, not over 200 bacteria per cubic centimeter, and the efficiency of a filter is to be estimated at 98 to 99%, it will be necessary to modify the quality of the water which is to be supplied to the filter, so that the number of bacteria which reaches it shall not be above 10 000. This should offer no serious engineering difficulty.

In regard to the color and odor which are not removed by filtration at Albany, it would seem that the chosen position of the intake may account, in part, at least, for the trouble. The "back channel" receives the drainage of the municipal gas-works, the waste accumulations which result from the sand-washing operations of the filters, and, not improbably, diluted sewage from Albany, which is carried up by the action of the flood tides.

It would seem that the original plan of an intake in the main channel of the Hudson was more favorable, from every point of view except that which takes into account the normal turbidity of the Hudson River water. Had it been found expedient to place the new intake in the main channel, and to provide added facilities for sedimentation or chemical precipitation, there seems to be no doubt that the operation of the filters would have been more uniform, and the quality of the effluent more satisfactory as regards bacteria, color and odor.

These are a few of the points which have been considered by the writer, after a review of the information which has been supplied. It is hoped that more information concerning the preliminary studies of the raw water will be forthcoming, and that additional data concerning the efficiency of the plant will justify its design and support the very high opinion with which the work is now regarded.

ALLEN HAZEN, M. Am. Soc. C. E. (by letter).—The discussion has brought out a large number of practical points, both in reference to the Albany plant and to filtration in general Mr. Hazen.

The results of operation given by Mr. Bailey are most gratifying, and the cost of operation, as exhibited in the comparison made by Dr. Mason, is very favorable, and shows an excellent organization of the work.

The water quantities given by Mr. Bailey are taken directly from the filter records. Some preliminary experiments have indicated that the coefficient of discharge assumed in computing the orifices was too small by about 5%, and that this amount should be added to the results. The value of the coefficient of discharge seems to be almost exactly the same whether the orifice is submerged or not. It is the intention to make more precise determinations, and afterward to have new and correct scales painted and substituted for those now in use.

Since the presentation of the paper a flood has occurred higher than any on record except that of 1857, which was a little higher. No damage was caused, but when the water exceeded the height of the overflow in the sedimentation basin, the river water entered it in that way. One of the pumps was kept in operation at a low rate to keep the pumping station dry. The operation of the filters was not interrupted or interfered with in any way.

In discussing the vaulting, Mr. Hill has made certain computations of quantities, and in doing so he has, apparently, divided the total amounts of concrete in the vaulting and in the flooring by the number of bays. In doing this, the writer thinks, he has overlooked the number of bays in the pure-water reservoir, and has also overlooked the fact that nearly half of the concrete in the floors was in the sedimentation basin and had nothing to do with the filters. He has also omitted the cost of the cement.

The figures for one section, 13 ft. 8 ins. square, corrected, and adding the price of the cement, are approximately as follows:

As executed:	
5.4 cu. yds. of vaulting, at $6.30	$34.02
4.85 cu. yds. of flooring, at $4.75	23.04
1.24 cu. yds. of brick work, at $9.67	11.99
	$69.05
As proposed:	
10 cu. yds. of roof slab, supporting column, floor and foundation, at $4.75	$47.50
Centering, 4 cents per square foot	7.47
Expanded metal	10.60
	$65.57

The 5.4 cu. yds. of concrete vaulting, per section, given above, includes the proportionate part of all special structures, and of the excess weight of the cylindrical vaulting near and over the walls.

Mr. Hazen. Without these, but including the manholes, the actual amount of concrete was only 5.1 cu. yds. per section.

The exact form of construction suggested by Mr. Hill was not considered, but several others of the same general type were studied before the plans were put in final shape. Some of these methods appeared very promising. The cheapening, however, depended upon a reduction of the thickness of concrete to less than 6 ins. While good results might no doubt have been obtained with concrete 3 or 4 ins. thick, reinforced with steel, there was no known precedent for its use under similar conditions. The Water Board was unalterably opposed to the use of any form of construction which could be regarded as in any degree experimental, and for this reason it was decided to use only types of construction which were well demonstrated. It was therefore necessary to postpone until another time a practical trial of the steel and concrete construction.

It would have been better to have covered the whole of the vaulting with a thin layer of sand before placing the silt and soil, and, particularly, to have surrounded the manholes with gravel. The draining would have been facilitated by this procedure, and the lifting of the manholes by frost would have been made impossible.

The estimate of the cost of vaulting per square foot, given by Mr. William B. Fuller, is a little greater than that given by the writer, recently, in discussing Mr. Metcalf's paper.* The difference arises from, first, the fact that Mr. Fuller has reckoned the cost of the vaulting upon the net filtering area, while the writer reckoned it upon the whole area covered; and, second, the fact that Mr. Fuller has included in the cost of the vaulting the cost of that part of the floor which he assumes to be due to its use as a foundation. The cost of vaulting, of course, is much the largest element of difference between the costs of open and covered filters, but it should be remembered that there are other points of difference, and that deducting the cost of vaulting from the cost of covered filters does not necessarily give the cost of open filters. Correct comparisons can only be made by examining corresponding designs for filters of the two types, using the same unit prices.

Mr. William B. Fuller and Mr. Fowler have suggested that covered filters have other advantages than protection against frost which may make their construction desirable, even in climates where covers are not necessary to prevent ice. This may sometimes be the case, but it should be borne in mind that, aside from the question of ice, there are distinct advantages and disadvantages arising from the use of covers. The writer does not propose to discuss this question at length; but, to show that this opinion is not unanimous, he will state that Dr. Strohmeyer, after making the extended investigations mentioned in Mr.

* *Transactions*, Am. Soc. C. E., Vol. xliii, p. 63.

Whipple's discussion, states that he has come to the conclusion that open filters have decided advantages over covered ones, quite aside from considerations of difference in cost. Mr. Trautwine also raises this question. Of course, in severe climates there is no question as to the necessity of covering filters. It is only where the winters are not too severe that the question arises. Mr. Hazen.

The sufficiency of the vaulting as a protection against cold has been tested during the past winter. Ice has formed to a thickness of 3 or 4 ins. immediately about the entrances to the filters, but it has been found possible to break up this ice and let it pass through the gates leading to the overflow chambers as the water is drawn from the filters before cleaning. Over the rest of the filters a skim of ice has formed occasionally, but this could be thrown aside during cleaning, and has not seriously interfered with the work.

Mr. Rafter is right in stating that the writer assumed that, on the whole, cracks are to be expected. It is a matter of common experience that small masonry structures remain entirely free from cracks. As the size of the structure increases, particularly if the masonry is comparatively light, the probability of cracks increases. It should be remembered that temperature contraction is only one of the causes of cracks in masonry. Cracks often occur through settlement, and at Albany six cracks were caused by the lifting of the walls by frost, due to the exposure of some of the work in an uncompleted condition. When a crack has once occurred it is not an easy matter to repair it so that the wall will be as strong as it was originally, and it then makes little difference whether it was caused originally by settlement, temperature or other causes.

When the plans were being drawn, the question of the bearing power of the foundations was considered quite seriously. The site was a soft marsh. Borings showed a hard material at a comparatively slight depth over the greater part of the area. When test pits were dug, a clay was found fairly hard as first exposed, but shrinking considerably on drying, and, if disturbed in contact with water, becoming very soft. Experiments were made by loading 1 sq. ft. of this material, and some settlements were observed. A part of the filters was to rest upon this foundation, a part upon rock, and a small part upon still softer clay. It was expected that there might be settlement in parts of the work. With this in view, an attempt was made, first, to distribute the weight over as large an area as possible, thereby reducing the probability of settlement; second, to load the whole foundation as evenly as possible, so that in case settlement occurred the whole structure would go down together, with the minimum damage; and, third, the possibility of settlement and of cracks was contemplated in designing all parts of the structures, and the endeavor was made to arrange the masonry so that it would suffer as little as possible in case

Mr. Hazen. of settlement. Provision for cutting off water coming through cracks was also made, as described in the paper.

When the excavation was made, the foundation proved to be much better than we had felt safe to assume from the character of the borings and test pits; and the fact that no measurable settlement occurred on any part of the filters must be attributed rather to the natural excellence of the foundations than to the design. In the pumping station some settlement actually took place, but the cracks resulting from it have not proved serious.

The cross-walls were built of brick instead of concrete, principally because the writer supposed that brick would be less likely to crack than concrete; and also because he believed that in case it did crack, the cracks could be repaired more easily. The writer's more recent experience with concrete, in this respect, has been quite favorable; and in a smaller plant, since designed and built, at West Superior, Wis., all the walls are of concrete. The bonding in these concrete walls was made as described in the paper, and the writer believes that in this way the joints between the old and new work are made substantially as strong as any part of the work. When cracks occurred at Albany, they went straight through the walls, and did not follow any of these joints.

To build masonry structures without cracks is certainly desirable. The writer believes, however, that a much larger proportion of the inferior work performed by filtration plants than is generally supposed has resulted from cracks in the masonry structures; and until the art of masonry construction is so far advanced as to make it quite certain that structures can be built without cracks, it is justifiable and necessary in designing filters, to consider the probable effect of cracks, and to take precautions against the damage which might result from them. Such precautions can be made entirely effective, and at a comparatively small expense. The writer believes that the design at Albany is such that, had there been many more cracks, or had there been some little settlement in portions of the work, the structures would still have maintained their stability, and would have continued to serve the purpose for which they were built.

In reply to Mr. Maignen's question, the sand-washing apparatus is not protected from the weather, and no sand is washed during the winter months. The dirty sand during this time is piled up in the court to be washed in the spring. It would, no doubt, be possible to protect the apparatus so as to wash in winter, but there is no objection to allowing the sand to remain until spring.

In regard to the thickness of the gravel layers, mentioned by Mr. Fowler and Mr. G. W. Fuller, the writer has constructed and observed many filters during the last dozen years, many of them experimental, and a smaller number for actual work. In many cases he has used gravel layers thinner than those at Albany. He does not remember

that he has ever used thicker layers in the aggregate, although in some cases the minimum over the tops of the drains may have been a little greater. He has had occasion to examine quite a number of the drains after they had been in use for considerable periods, and in no case has he been led to think that thicker gravel layers would have been desirable. Thin gravel layers, of the right sizes and carefully placed, are, in his opinion, quite as good as thicker ones. Much thicker layers, carelessly placed, or not of the right sizes, are entirely inadequate. It is cheaper and better in every way to use comparatively thin layers, and to take the trouble to make them right, and have them effective, than to use thicker layers and depend to a certain extent upon chance for the results. Mr. Hazen.

The methods adopted by the contractors at Albany for screening and placing the gravel were particularly satisfactory. There has not been the slightest evidence of incomplete support, nor does the writer consider that there is any danger that the sand will get into the gravel so as to injure the filters.

In this connection, it may be stated that the coarse gravel over the drains, and particularly about the joints, was placed with unusual care. The workmen were instructed personally in the methods to be adopted in this part of the work. They quickly became skilful at it, and were faithful in carrying out their instructions.

Mr. Fowler mentions the clogging of the holes in the inlet pipes to the sedimentation basin. In practice, it is found that these holes are stopped up, to a certain extent. The appearance of the outlets is not very much changed until a considerable proportion of the holes is stopped. Practically, the rate of pumping always exceeds the capacity of the holes, and water is always flowing over the tops of the pipes. It is necessary to clean the holes at intervals, and this is done with a broom, by a man in a boat. The cleaning is done very quickly. It can be done at any time when the pumps are stopped, or, when they are running, by shutting off the inlets in rotation. The inlets have actually been cleaned at intervals of a week or two, and the labor required was very slight.

At another time, the writer would put the inlets somewhat farther away from the bank. With this done, it would be possible to leave the perforated pipes in position during the winter. As it is, the spray from them builds ice upon the bank. They were actually taken off soon after the commencement of cold weather. This is not a very important matter as they are easily taken off; there is probably no necessity for aeration during the winter, and, with the lower lift, a little coal is saved.

The question raised by Mr. Fowler as to the size of individual filters and the number of beds in a plant, is an important one. It should be remembered, however, that while the convenience of operation is generally increased by increasing the number of beds, the cost of construction per unit of area is also increased, and it may often be

Mr. Hazen. found best to sacrifice something of present convenience to economy of construction ; or, in other words, with a given sum of money, to build a larger area of large beds, rather than a smaller area of small ones.

The comparisons of populations upon several rivers, mentioned by Mr. Trautwine, are extremely interesting. It should be noted, however, that the figures given by the writer for the Hudson River include only the populations of cities, towns and villages, and, as stated in the paper, do not include the rural population. The rural population, obtained by going over the water-shed by counties, and deducting the populations of cities, towns and villages given in the table, amounts to about 30 per square mile, for 1890. The rural population does not change rapidly, and a close approximation to the total population at the various dates would be obtained by adding 30 per square mile to the figures for the urban population given in the table.

The methods of distribution of the cost of various parts of the plant, given by Mr. Trautwine, are not entirely clear to the writer. The figures given in the paper were obtained by going over each item somewhat carefully, and allotting the proper proportion to the structures indicated. The final results were given in round numbers, because of the impossibility of deciding the exact points of division between the various parts mentioned.

The question raised by Mr. G. W. Fuller, as to the quality of the raw water, has been already answered in part by Mr. Bailey in his discussion. Data upon these points are rapidly being accumulated at the laboratory of the works, and much more complete results will be available after a little while. The data at hand at the time of the preparation of the paper were so meager in comparison with those now being obtained, that it was thought best to let the matter stand until it could be discussed more adequately.

The general character of the water, as regards muddiness, is between the very clear waters of most New England streams, and the turbid waters of the Middle States, although it resembles the former more than the latter. Generally, the raw water is comparatively clear, but very muddy water is obtained occasionally, especially from the Mohawk, which drains a clayey country from which the water is quite muddy. The fluctuations in turbidity are much less rapid at Albany than in some other places, and when the water becomes turbid it remains so for several days. The greatest turbidity yet observed is such that a bright platinum wire 0.04 in. in diameter can be seen through only 1 in. of water.

In reply to Mr. Whipple's question, Bleecker Reservoir, which is perhaps the most important of the distributing reservoirs, has recently been thoroughly cleaned. The laboratory in connection with the filter plant will make studies of the vegetable growths which occur in the reservoirs, but it is too early to draw any conclusions upon this point.

Mr. Hazen.

The odor due to gas waste, mentioned by Dr. Mason, first occurred after the paper was prepared. The odor was offensive on only one occasion during the fall, resulting from a combination of extremely low water, spring tides, and a south wind. Dr. Mason has indicated the proper solution of this problem, and the gas company has made arrangements to build a drain to the Patroons Creek sewer, so that this material, instead of being discharged into the back channel, will be discharged into the river below the southern end of the island, where the opportunities for dilution will be much greater. It is also to be hoped that improved methods at the gas-works will reduce the amount of this highly objectionable material to be discharged into the river. It should be understood that this odor was due to material discharged into the back channel at a point near the intake, and carried up to it in a comparatively concentrated condition, and not to material which had become mixed with a considerable portion of water flowing in the river.

In reply to Mr. Soper's questions, the Prospect Hill or high-service reservoir, holding 7 500 000 galls., receives filtered water only, and its capacity is such that the water remains in it probably from one to two days. The Bleecker or middle-service reservoir, holding 30 000 000 galls., receives water by gravity from Rensselaer Lake in varying quantity, according to the supply, and the balance is filtered water. The relative proportions of the two supplies vary widely at different times. The Tivoli or low-service reservoir, holding 19 000 000 galls., is generally supplied by gravity, but water is let down to it from Bleecker reservoir whenever the supply falls short.

Regarding the standard of bacterial purity mentioned, as given in the rules of the German Board of Health, it should be remembered, in the first place, that this standard was purely provisional, and in the second place, that the number of bacteria is determined by growing on plates at a temperature of 20° Cent. for 48 hours. The writer is informed that current American bacteriological practice requires the growth of the plates for a considerably longer period than this, and that studies as to the best composition of the nutrient gelatine, etc., have resulted in the adoption of a procedure by American bacteriologists which gives much higher numbers than the German procedure.

It should be further noted that the bacterial efficiency of a filter is generally low at starting, and increases with use; and it is to be confidently expected that the bacterial efficiency of the filters will be greater during the coming year than that indicated by the figures given by Mr. Bailey.

Mr. Soper speaks of the removal of the bacteria by sedimentation. The writer believes that the improvement of water in this manner depends almost entirely upon the character of the sediment carried by it. Many western streams carry much sediment, to which the bacteria are

Mr. Hazen. apparently attached, and when the sediment goes to the bottom the number of bacteria in the water may be greatly reduced. The Hudson River water at Albany generally carries but little sediment, and holding the water for 24 hours in the sedimentation basin does not diminish materially the number of bacteria in it. It is the writer's opinion that, for the conditions at Albany, about 24 hours represents the economical limit of subsidence, and that no improvement in the water, commensurate with the increased cost, would be obtained by detaining it for a longer period.

Regarding other methods of preliminary treatment, it is undoubtedly true that the color could be removed by the addition of from 1 to 1½ grains of sulphate of alumina per gallon, and the alkalinity of the river water is probably such that this amount could be used without injurious results. It is believed that the present plant contains all the appliances necessary for the application of this procedure, except the tanks for dissolving the chemical; but it is not thought that the advantages to be obtained would be commensurate with the cost, and with the possible danger from the use at times of excessive amounts of coagulant. It should be noted further that it does not necessarily follow that the same percentage of bacterial efficiency will be obtained by filtering a water from which many of the organisms have been removed by preliminary processes. The contrary has sometimes been observed, when the filtration of a partially purified water has given effluents no better than could be obtained by a similar filtration of the raw water.

Regarding the position of the intake, the back channel has received the drainage of the gas-works, and the dirty water from the sand washing. Arrangements have been made to remove the discharge from the gas-works to a point below the back channel. None of the city sewers discharges into the back channel, the outlet farthest up stream being the Patroons Creek sewer, which enters the river a short distance below the lower end of the island. The writer believes that it is not true that much less turbid water is obtained from the back channel. At low-water stages the turbidity is substantially the same at all points. At flood stages the dike is overtopped, and the waters in each channel are identical. The advantage of the back channel consists, not in the sedimentation which takes place in it, but in the greater time required for the sewage-polluted water to reach the intake. This delay allows some of the bacteria to die, and it is in this way that the better bacterial condition of the water is to be accounted for.

The relative characters of the waters in the back and main channels are observed at frequent intervals, and when the construction of the intake to the main channel comes up again, very much more full and satisfactory data will be at hand as to the character of the water which could be secured from it.

Vol. XLIII. JUNE, 1900.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

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No. 873.

THE EXACT DESIGN OF STATICALLY INDETERMINATE FRAMEWORKS. AN EXPOSITION OF ITS POSSIBILITY, BUT FUTILITY.

By FRANK H. CILLEY, S. B.

PRESENTED NOVEMBER 15TH, 1899.

WITH DISCUSSION.

INTRODUCTION.

In the theoretic consideration of all material systems it is customary, in fact necessary, to replace them by more or less equivalent but simpler systems. This alone makes their analytic examination possible, not to say practicable; and the results thus obtained, although applying strictly only to the ideal constructions of theory, throw much valuable light on the workings of the actual material systems.

But so accustomed have we become to this procedure that we often forget that between the theoretic results and the actual conditions there must always exist a divergence, greater in proportion as the difference between the imaginary system and the real system is the greater. This neglect leads to two serious faults: *First*, in the consideration of existing systems, replacing them by theoretic systems, which insufficiently represent them, and applying the conclusions thus derived as though they were practically exact: *Second*, in the

creation of new systems, constructing them so differently from the ideal systems on which they are based that the actual conditions are quite unlike those supposed. This has given rise to many strange fallacies, has held back practice and has discredited theory.

Frameworks furnish excellent illustrations of this, for the ideal framework has always been a far simpler structure than the real framework, and its theory correspondingly different from actual conditions. Whereas the ideal framework consists of a group of straight members so joined at their ends as to move freely about the points of connection, loaded only at these points, and—neglecting the effects of dead weight and wind pressure—subject, therefore, only to direct stress of tension or compression in the members, actual frameworks always have joints more or less stiff, their members are frequently joined at other points than their ends, and they are loaded at other points than their joints, thus introducing flexure of the members, as well as direct stress, as a result of the application of loads.

But it has so long been customary to overlook these divergencies between theory and practice, to gratuitously assume them of no consequence, that we now, as a matter of course, figure our simple theoretic system and then apply the results to our complex actual system, without a doubt, apparently, that this application is not only justifiable, but practically exact. This has resulted in the perpetuation of defective construction and the fallacy that increased strength and stiffness, with the same amount of material, is to be obtained by stiff joints, broad members rigidly attached where they pass one another, continuous members, etc. To this day, some eminent engineers uphold these as virtues. And, on the other hand, this has resulted in indifference as to the variations between theory and practice which actually exist, and a neglect to develop, and still more to apply, more exact parallelism, both in theory and practice. In America a partial attempt to fulfill these suppositions of the ideal framework by the use of pin connections was made long ago, yet we continue to use continuous chords, and floor systems, and many riveted connections of which the theory takes no account. In Europe a few attempts have been made to calculate closer theoretic parallels to actual structures, and these more exact calculations have shown how very considerable were the neglected elements of stress and the importance of eliminating these "secondary" stresses as they are called; but these calcula-

tions are very recent, and are extremely few in number, for they are very laborious.

The net result is, that not only do theory and practice diverge to an important extent, but that this divergence is given practically no attention. This is not as it should be, and we may look forward to considerable improvement in this respect in the near future. The writer believes that a much closer approach of the actual to the ideal framework is not only practicable, but desirable; and, on the other hand, he considers more exact calculation, that is, calculation of the structures more nearly as they are, a requisite to progress. Expensive mechanical devices should not be used in the effort to approach more nearly ideal construction where no corresponding practical advantage can be shown, and laborious calculations are out of place where the limits of error of simple approximate calculations are known and can more cheaply be taken into account through liberality in construction. But what may be insisted on is that either the actual conditions shall be taken into account through more exact calculation, or that no pretence of accuracy shall be made where there has been none, and that the divergences between the theory used and the construction actually carried out shall be allowed for liberally.

The object of this paper is not to consider the more exact calculation of actual structures, but rather to examine certain questions in connection with ideal frameworks, of the greatest practical importance because at the foundation of actual designs. It deals with statically indeterminate ideal frameworks, that is, ideal frameworks as previously defined, in which not all the members are essential, or, in other words, in which some members are superfluous. What are the conditions involved and how may such frameworks be exactly designed; what are their qualities, and what their merits and demerits? Such is the matter which this paper seeks to consider. Its purpose is not to give instruction in methods, but simply to indicate the principles involved, to point out some fundamental but not generally known facts, and to discuss broadly, with the aid of a few simple illustrations, the bearing of these facts on practice. And let not such discussion of ideal construction be underrated and regarded as of no value in connection with actual structures, which all, more or less, diverge from the ideal, for it must be remembered that the ideal here considered is the basis on which all actual practice is founded.

The use of statically indeterminate frameworks, in the sense here mentioned, has been exceedingly common in Europe, where it has been, and is even now, advocated extensively by the most eminent engineers. Continuous bridges, two-hinged and hingeless arches, arches supporting continuous trusses, and their inversions of suspension type, not only are met frequently in existing structures, but are used frequently in new projects. And there are indications that here in the United States these constructions are gaining a certain hold, as shown by the erection of certain recent arches and suspension bridges, and above all by the proposed huge suspension bridge at New York. Moreover, since most frameworks are actually space-frameworks, formed by uniting several plane frameworks, and this connection is made without consideration of statical determination, most frameworks—as bridge trusses with their connecting wind bracing—are actually indeterminate, whether or not their component plane frameworks are statically determined when alone. This gives great prominence to the subject of the construction of frameworks with superfluous bars, and makes a clear understanding of them exceedingly important. Is it advisable to use such structures rather than the much simpler "just determined" construction generally advocated in America? Which is the better, and why?

In the course of the more mathematical examination which will follow, the answer to this question will present itself, but it is advisable here to review briefly certain facts.

European engineers have always been the principal advocates of indeterminate construction. They have insistently claimed for it the virtues of greater economy, strength, safety and stiffness. Practically, no demonstration of these claims has been forthcoming, except, perhaps, proof that the continuous girder possessed these advantages as compared with equal free spans; but this comparison was unfair, and the proper rejoinder to it was eventually given by the cantilever. Grudgingly conceding its equivalence, European engineers continue to claim, without demonstration, the superiority of the remaining types, particularly of the two-hinged and hingeless arches over the three-hinged arch.

These claims have not been wholly uncontested. As early as 1876, Maurice Levy, in Paris, insisted strongly on the universal economic superiority of frameworks with no superfluous bars. He supported his contention by general reasoning, and demonstrated

mathematically, that in a certain exceptional case with superfluous bars the economy was no greater than without such bars. But little attention seems to have been paid to Levy's statements, possibly because they were not accompanied by any general mathematical proof. The subject, from this point of view, seems to have been dropped entirely from consideration, and the writer, in his studies in Paris, Zurich and Berlin, perhaps the most notable of the European engineering schools, has found no further mention of it. In these schools statically indeterminate structures are universally taught and advocated, being regarded, apparently, as more difficult, but better, than the simpler, determined structures. Beautiful methods of calculation have been developed for the more complex structures, and the professors, possibly fascinated by the charms of these computations, gave them great prominence, and fostered among the students a predilection for difficult types, without making any study of the question, whether or not there were real advantages in their use to offset their difficulties.

Feeling convinced that, while the knowledge of methods of calculation of difficult structures was exceedingly valuable because of their frequent occurrence, and because of unavoidable occasions for their use, the encouragement, even preference, given to the use of such construction was mistaken, the writer, fully informed as to European ideas and methods, undertook a general investigation of the comparative merits of frameworks with and without superfluous bars. The result was the discovery of some curiously unknown, although most fundamental, facts concerning the design of frameworks with superfluous bars and the distribution of stresses therein, together with a somewhat general demonstration of the economic superiority of statically determined over indeterminate construction. In a simple and popular article published in the *Technology Quarterly*,* for June, 1897, these results were set forth, but it was desired to present the matter, in a somewhat more mathematical and complete form, directly to the engineering public, for whom it would have a particular interest.

Consequently, the present article has been written. In it the simple illustrations and much of the matter of the preceding article have been passed over with a mere reference, the subject being treated somewhat more generally, and much space has been devoted to some illustrations of selected, extremely simple cases of exact design, with

* Published by the Massachusetts Institute of Technology.

the purpose of demonstrating the impracticability, the futility, even the folly of attempting such designing in practice, rather than furnishing instruction in its use. It is desired simply that those who read this, by the inspection of the amount and difficulties of the work as exposed in the illustrations, may form some adequate notion of why the exact design of frameworks with superfluous bars is not to be thought of, and, therefore, why such design in practice, in the future as in the past, will be made by methods of trial, which can only give at best, after infinite labor, most imperfect and uneconomic results. But it is desired that they may examine with care the results obtained in the illustrations, to convince themselves that even with exact methods, giving the most perfect possible designs, and when neglecting temperature effects and many other serious objections, the use of superfluous bars furnishes no sensible advantage, while, including these inevitable accompaniments of indeterminate construction, it is in all respects inferior to the simple determinate construction, taught and advocated in our schools, and with which American bridge engineers are perfectly familiar.

To those whose interest and leisure permit, more careful perusal of the illustrations is recommended, and in particular the working out of similar examples. Such work would form a valuable supplement to the little that the writer is here able to offer. And further, he would suggest the careful analysis of some notable existing indeterminate structures, and their comparison with designs of parallel statically determined structures. The results will inevitably be surprising in their demonstration of the sad deficiency of such indeterminate structures, judged by the standards of good proportioning insisted on in all statically determined designs. American engineers may in the future devote their attention to the determination of the best statically determined forms, to the development of statically determined space-frameworks and to the perfection of details intended to ensure the usually assumed ideal simplicity of stress, with the assurance that they are on the sole road to the best designs.

The writer further ventures to hope that structures now in contemplation but not commenced, projected in statically indeterminate form, will be reconsidered, and that more economic statically determined forms, which certainly exist, will be determined and used in their stead.

The Exact Design of Statically Indeterminate Frameworks.

In an ideal framework, since it is composed only of members joined at their ends, freely movable at these joints and loaded only at the joints, we are concerned only with the equilibrium of forces acting at points—the joint centers. Therefore, the conditions at each joint are three in number, that the resultant forces along three rectangular axes shall be zero. If a framework have n joints this furnishes $3n$ equations of condition which must be satisfied. Of these $3n$ equations but $3n - 6$ are independent equations involving the inner forces alone, since they must together satisfy six conditions of equilibrium among the outer forces acting on the framework, including in these outer forces the elements of the reactions as well as the loads. In general, for the stability of the structure, these elements of the reactions must be at least six in number. Let their number be r, and the number of members of the framework or bars be m. Then, for stability, $m + r$ must at least equal $3n$. But $m + r$ may perfectly well exceed $3n$, in which case the framework becomes statically indeterminate, because the $3n$ conditions furnished by statics are insufficient to determine the $m + r$ unknown bar stresses and reaction elements.

If the framework were statically determined we should have $m + r = 3n$, the form of the framework would be just determined through the lines of its figure, it would be "just stiff"; remove any bar and it would become a mechanism; alter slightly the length of any bar or the position of any support and the figure of the framework would be altered slightly without further consequences; no stress would exist in the members, except through loads; temperature changes, whether uniform or varying, would change slightly the form of the framework without affecting it otherwise; the stress in each member due to any load would be calculable easily and without regard to its own material and its elastic qualities or those of its neighbors; each member could be designed readily and exactly under any given series of loadings to satisfy almost any given conditions, for the dimensioning of each member would be practically independent of that of all others; and, finally, all displacements of the joints of the framework due to stresses or temperature changes would be just defined through the corresponding changes of the lengths of the members, and would be easily calculable.

If to such a framework we were to add further bars, the joints

remaining as before, so that $m + r > 3n$, the framework would become overdetermined; some of the bars could be removed, and the framework would still be stiff; alter slightly the length of any of these bars or the position of a support, and the framework would be put under stress; stress without loads might, therefore, exist, and temperature changes might cause very considerable stresses; the stresses under given loads could only be calculated by long and laborious application of the equations of elasticity, in which not only the dimensions of each member but its material and elastic qualities would have to be taken into consideration; the stress in a single member could not be determined independently of other members; the design of a given member might be affected greatly by the design of many other members, and it would be very difficult, if not impossible, to satisfy accurately given conditions under a series of loadings; finally, minute changes in the lengths of some of the bars, or slight yielding of the supports, might change materially the distribution of stress, so that it would be exceedingly difficult in practice to obtain even approximately any stress distribution which in the design was determined upon, and to maintain such stress distribution in the face of any slight yielding of joints and supports due to changing loads, climate and other external, or, perhaps, also internal, causes.

Such are the structures whose exact design we are about to consider. In the foregoing we have regarded them as formed by the addition of further members to "just determined" frameworks, although they have rarely been thus conceived. This manner of regarding them is most serviceable in connection with their calculation, but the course of evolution has rather been the derivation of "just determined" forms from pre-existing indeterminate forms through the dropping of superabundant (or superfluous) members.

The recital of the more obvious characteristics of the indeterminate frameworks, just made, would seem alone a sufficient condemnation of their use in practice, and further facts, developed in the later analysis, will serve to confirm definitely this view. The statical analysis of frameworks with superfluous bars shows them to be inferior in economy and strength while not superior in stiffness to the best parallel designs without superfluous bars.

The objectionable characteristics of indeterminate structures have long been known, but hitherto have been regarded as offset by certain

supposed virtues of economy and stiffness which have been ascribed to them. Yet the correction of these mistaken beliefs through analysis, which is the object of this paper, will destroy the prestige of indeterminate structures only in part. They have, and will long continue to have, the support of many able engineers, whose experience, having been largely with such structures, prejudices them in their favor and disables them from appreciating and weighing their defects clearly. Following the common human device of seeking in reason justification for beliefs rather than through reason establishing belief, some of these engineers will make much of what analysis has not yet defined and will strive to make of the unknown, and, for the present, perhaps, unknowable, not simply a defence, but a justification of the continuance of their beliefs. We shall find them taking to ground so difficult that the theorist, at least as yet, is unable to follow and settle the matter decisively, and maintaining that, in general, the problems connected with frameworks are not static, but dynamic; that inferiority, demonstrated for static loads, no longer is true of moving loads; and that the introduction of vibration in addition to static deflections wholly alters the case and leaves indeterminate structures, after all, masters of the field.

But it must be remembered that these will only be claims, not proofs. They can only be supported by general reasoning, and can be effectively answered in the same fashion. Experience has not demonstrated that dynamic defects are necessary accompaniments of static determination, and all not proven by analysis remains disputable. Let us only not be outfaced by displays of erudition or assertions of authority, and we will speedily perceive that in appeals to unconsidered and complex phenomena (the writer has heard brought up the speed at which stress travels and the consequent effect of the time of loading on stress distribution), we have only to do with appeals to ignorance, which can hardly be regarded as adding strength to the arguments of those who make them.

The writer does not wish by the preceding to belittle the importance of dynamic phenomena. In due time and in their order they should be and doubtless will be studied and given proper consideration. But first we must study and understand the static phenomena which are at their foundation, the truths of which they will not change but only add to.

In the following paper no attempt is made to treat mathematically

other than the static side of the question. And to begin with we may point out a fact, hitherto almost wholly disregarded, and which makes indeterminate frameworks worthy of the name far more than does the fact that they are statically indeterminate.

We have already noted that in indeterminate frameworks the number of unknown quantities is greater than the number of equations of condition furnished by statics, that is $m + r > 3n$. We may conveniently regard the framework as possessing as many superfluous bars as $m + r$ exceeds $3n$. Let the bars i ($i = h$ to m inclusive) be selected as the superfluous bars; then the remaining bars e ($e = 1$ to g inclusive) form with the supports a statically determinate framework. Let S_{e_0} be the stress in any bar e of this statically determined framework if this framework alone carried the loading, let $S'_e{}^{(i)}$ be the stress in this bar e which a tension of unity in one of the superfluous bars i would cause in e if there were no external load and if of the superfluous bars the bar i alone were in action, and let S_i be the actual stress in the superfluous bar i when the given load is applied to the actual framework with all its superfluous bars acting. Then the actual stress S_e in a bar e is evidently given by the formula:

$$S_e = S_{e_0} + \sum_{i=h}^{i=m} S_i S_e'^{(i)}$$

Now—and here is the commonly neglected consideration—as far as the laws of statics are concerned, only the S_{e_0} and the $S_e'^{(i)}$ are fixed and determinate, while the S_i may be anything; they are truly indeterminate and not, as commonly stated, fixed by certain auxiliary equations of elasticity. What does fix them in a structure with given elastic constants and given bar sections is, under given conditions of temperature, the precise lengths of the bars. But, as a matter of fact, in any existing structure these lengths of bars are never known with anything like exactitude, as will be evident when one considers that, with a modulus of elasticity of 30 000 000 lbs. per square inch, a bar of steel 10 ft. long, whose length is not known within a thousandth of a foot, would not have its intensity of stress known within $\frac{30\,000\,000}{10 \times 1\,000} = 3\,000$ lbs. per square inch, or within about a third of its probable working stress. Designers and computers have passed over this difficulty, often, if not generally, unconsciously, by the simple assumption, unwarranted by any corresponding precautions in construction or knowledge of existing structures, that, at a certain

temperature and with a certain loading, a certain state of stress exists throughout the structure. Usually, this takes the simple form of the assumption that the structure, when free from all loading and uniformly at the mean temperature, is free from all stress. Then truly the equations of elasticity define the stresses under all other conditions of loading and temperature, but it is this far from warranted assumption and not the equations of elasticity which removes the indetermination.

Since the stresses in the superfluous bars are not fixed by equations of elasticity, but by their lengths, we evidently have it in our power—in designing at least—to give these bars any stresses we please under a given loading by giving them suitable lengths, and this without in any way altering the figure of the framework (because the changes of length of the superfluous bars required in all practical cases would be very small), and without altering the sections of the bars, that is, the design as it is ordinarily defined. If these superfluous bars be k in number this gives us a k^{tuple} degree of freedom in the design of such a framework, and the same degree of indetermination in the computation of such an existing framework. Unlike the case of a statically determined framework where the distribution of stresses due to a given loading is invariable, that distribution may here be varied in an infinity, a k^{tuple} infinity, of ways. In designing, we may select any one of these at will, then design the cross-sections to suit, and finally compute very exactly the corresponding lengths of the bars. It will remain to construct them exactly as computed to realize the designed stress distribution. But if the structure be an existing one given us to compute, who can say which one of this infinity of possible stress distributions is that actually existing under given conditions? The problem is actually indeterminate and in increasing degree as the number k of superfluous bars is greater.

We are now prepared to consider the subject of the title, "The Exact Design of Indeterminate Frameworks." But first, what is meant by "exact" design? Any one familiar with the design of indeterminate frameworks is aware that, unlike the design of determinate frameworks (if we neglect the estimation of dead weight), the process is essentially tentative instead of direct. That is, given the figure of the framework, certain bar sections are assumed, calculations of the stresses corresponding made (on the assumption of no stress under no load at mean temperature), the divergences from the desired con-

ditions as to intensity of stress noted, proper corrections of the sections guessed at, the computations all repeated, new corrections guessed at, then new computations made, and so on, repeating again and again all the work until the result approaches that desired sufficiently to be acceptable. But as an alteration in the section of any one bar affects the stress not in itself alone, but in many other bars also, this process of correction is not simply exceedingly laborious, but, as every designer of such structures knows, very uncertain; and after infinite labor the result obtained is usually one, the deviations of which from the set conditions of maximum allowable intensity of stress are such as would not be tolerated by any good engineer in a statically determined framework.

By "exact" design is meant: Expressing in equations the conditions as to intensity of stress to be satisfied by the solution under the given loadings, and then by direct solution of these equations, without any tentative process (other than that mathematically necessary to solve the equations), obtaining at once and exactly the desired design, *e. g.*, the cross-sections and the bar lengths. It will be appreciated that such an exact method, apart from any question of practical advantage over the old tentative method, would be likely to throw most instructive light on the properties of indeterminate designs, which the old method could only feebly give through statistics, or practically not at all; and the object in bringing forward this "exact" method is simply to throw such light; for, apart from the fact that the writer reaches through this method the conclusion that the best designs of indeterminate frameworks are necessarily inferior to those of determinate frameworks which necessitate no such methods, the illustrations to follow will, among other things, show conclusively that the mathematical difficulties of the exact method render its application impracticable in general.

To begin with, we will consider the simple but very instructive and important case of design for a single given loading, subject to the condition of a given intensity of stress in each bar under that loading. This case will be somewhat briefly treated, since, in an earlier article,* it has been fully considered and illustrated.

As pointed out previously, an infinite variety of stress distributions under a given loading are all equally possible, regardless of the cross-

* *Technology Quarterly*, June, 1897. "Some Fundamental Propositions Relating to the Design of Frameworks."

sections of the members. Having selected at will any one of these stress distributions, we next design the cross-sections of the various members to carry the stresses thus assigned them with the assigned intensities of stress, using column formulas or not as we choose. It only remains to determine the lengths of the bars necessary to ensure the supposed stress distribution. This we may do as follows:

In the given figure of the framework the exact length of every line is geometrically determinate if the data for the figure are sufficient, and this we suppose. Thus, to start with, we have what may be called the figure length L_f (where $f = 1$ to m) of every one of the m bars. Now, we know that, supposing the points of support fixed, the exact position of each joint is geometrically determined by the exact lengths from center to center of joints of the series of bars e ($e = 1$ to g), which together form a statically determined framework with the same joints (see page 573). The lengths of the bars i ($i = h$ to m), which we have termed the superfluous bars, are therefore geometrically deducible from the lengths of the bars e. Let us assume the actual unstrained lengths from center to center of joints of the bars e, at standard temperature (lengths which we may in general denote by l), to be the figure lengths, or that $l_e = L_e$. On this basis we now have to determine the actual unstrained lengths l_i from center to center of joints, at the given standard temperature, of the superfluous bars i, so that, under the given loading, each bar shall have the stress we have already determined for it.

These lengths l_i will not in general be the same as the figure lengths L_i, which we may regard as exactly defined through the L_e and therefore known. For, under the given loading the bars e are subject to the stresses S_e and, therefore, no longer have the lengths $l_e = L_e$, but have the strained lengths $L_e + \frac{S_e l_e}{E_e A_e}$, where E_e and A_e are the modulus of elasticity and the section area, respectively, of the bar e. And the distances from center to center of the joints connected by the superfluous bars i, that is the strained lengths of the superfluous bars i, are at the distances defined by and resulting from these strained lengths $L_e + \frac{S_e l_e}{E_e A_e}$. If we determine these strained lengths of the superfluous bars and then design their actual lengths l_i, so that the strained lengths shall correspond to their supposed stresses S_i, that is, so that the strained lengths shall be $l_i + \frac{S_i l_i}{E_i A_i}$ then we shall evi-

dently have so made our design as to secure our supposed distribution of stress. These strained lengths of the bars i might be directly determined by ordinary geometrical principles from the strained lengths $L_e + \frac{S_e l_e}{E_e A_e}$ of the bars e, but, since we have all the figure lengths L, and we may suppose the changes from these to be very small, it is preferable to proceed as follows, making use of the principle of virtual work. Let the bars e suffer any small elongations Δ_e from any causes whatsoever, then the consequent approach of any pair of joints not connected by a bar e will evidently be the same as the work that would be done by two equal and opposite forces of unity, each acting at those joints toward each other, through this approach, and this external work of the pair of forces unity would in turn be equal to the internal work the stresses balancing them in the bars e would do through the elongations Δ_e of the bars e. Now, these stresses in the bars e, balancing such a pair of equal and opposite forces of unity between the joints to be connected by the superfluous bar i, we readily see are the quantities ${S'}_e^{(i)}$, already noted on page 362, so that the approach of the joints to be connected by the bar i, due to the elongations Δ_e of the bars e we at once see would be

$$\delta_i = \sum_{e=1}^{e=g} {S'}_e^{(i)} \Delta_e *$$

In our case, the Δ_e we suppose to be the elongations due to the stresses S_e, that is, we have $\Delta_e = \frac{S_e l_e}{E_e A_e}$, whence follows:

$$\delta_i = \sum_{e=1}^{e=g} \frac{{S'}_e^{(i)} S_e l_e}{E_e A_e}.$$

But $L_i - \delta_i$ is the strained length of the superfluous bar i which is also, as we have noted, $l_i + \frac{S_i l_i}{E_i A_i}$, so that we find for our desired unstrained length of the superfluous bar i the value

$$l_i = L_i - \left(\sum_{e=1}^{e=g} \frac{{S'}_e^{(i)} S_e l_e}{E_e A_e} + \frac{S_i l_i}{E_i A_i} \right)$$

$$= L_i - \sum_{f=1}^{f=m} \frac{{S'}_f^{(i)} S_f l_f}{E_f A_f}$$

(where we must note that the ${S'}_f^{(i)}$ are all zero for all values of f from h to m except i, when ${S'}_i^{(i)} = +1$).

* For a much fuller account of the principle here used and its applications, see the article "On the application of the Principle of Virtual Velocities to the Determination of the Deflections and Stresses of Frames," by Professor George F. Swain, in the *Journal* of the Franklin Institute, April, 1883.

The foregoing neglects any temperature changes, but were there such they would be accounted for by putting $\Delta_f = \frac{S_f l_f}{E_f A_f} + \tau_f t_f l_f$, where τ is the coefficient of expansion and t the increase in temperature above the standard used.

This determination of the exact values of the bar lengths, which ensures the supposed stress distribution under the given loading, completes the design, but there still remains a matter of interest for consideration, the "primary" stresses or stresses remaining in the bars after the loading is removed. That such primary stresses will in general exist is evident from the fact that the superfluous bars have lengths l_i differing more or less from the figure lengths L_i, which would correspond to a condition of no stress at standard temperature with no load. Let us denote the primary stresses by (S); then, reasoning as before, the primary stresses (S_e) in the bars e determine the distance between joint centers of any one of the superfluous bars i to be

$$L_i - \sum_{e=1}^{e=g} \frac{S'_e{}^{(i)}\,(S_e)\,l_e}{E_e A_e}.$$

But this is the length of the bar i under its primary stress, or the length $l_i + \frac{(S_i)\,l_i}{E_i A_i}$; whence we obtain the equation

$$l_i + \frac{(S_i)\,l_i}{E_i A_i} = L_i - \sum_{e=1}^{e=g} \frac{S'_e{}^{(i)}(S_e)\,l_e}{E_e A_e}$$

which, adopting the previous notation, we can express in the form

$$\sum_{f=1}^{f=m} \frac{S'_f{}^{(i)}\,(S_f)\,l_f}{E_f A_f} = L_i - l_i.$$

Now, for each of the k superfluous bars i we can write out such an equation, each of which contains the primary stresses (S) as the only unknowns, and they appear only in the first degree. But the primary stresses (S_e) in the bars e we know are expressible in terms of the primary stresses (S_i) in the superfluous bars i through the linear equation $(S_e) = \sum_{i=h}^{i=m} (S_i)\,S_e'^{(i)}$ (see page 362), so that, making these substitutions in the k preceding equations, they become simply k linear equations between the k unknown primary superfluous bar stresses (S_i). These equations solved, from the now known (S_i) we determine the (S_e), and so all the primary stresses. We have only to note that the primary stresses thus found must be less than those occurring under the given loading, for the design to be acceptable.

So much for the design; now consider certain consequences. Suppose the section areas were designed by the simple rule $A = \frac{S}{\sigma}$ where σ was the working stress allowed. Then A being simply proportional to the stress S, each section area, and, therefore, the volume of material of each bar, would be simply proportional to its stress under the given load, and the total volume would be expressed simply by

$$V = \sum_{f=1}^{f=m} \frac{S_f}{\sigma_f} l_f$$

But the stresses S_e in the bars e are expressible in terms of the stresses S_{e_0}, (which would occur in them under the given loading if there were no superfluous bars), and of the arbitrarily variable superfluous bar stresses S_i, through the linear equation $S_e = S_{e_0} + \sum_{i=h}^{i=m} S_i \, S_e'^{(i)}$ (see page 362), so that ultimately the volume is expressible in the form

$$V = C + \sum_{i=h}^{i=m} c_i \, S_i$$

where c_i and C are constants. Now, were c_i and C always the same, whatever the S_i might be, V would take all values from $+\infty$ to $-\infty$ by suitably varying the S_i. But, actually, we are limited by the condition that $\frac{S_f}{\sigma_f}$ must always remain positive, so that, if by any variation of the S_i the stress in any bar becomes zero and changes to stress of the opposite kind, its working stress σ at the same instant is changed in kind, which not only involves a change in the sign of σ, but also a change in its value if different working stresses in tension and compression be used. Thus, V is expressed by a discontinuous linear function of the k independent variables S_i, the discontinuity of which occurs only when some bar stress becomes zero. Such a function has a minimum only at a point of complete discontinuity, that is, where the variation of the arbitrary stresses S_i causes k bars to pass simultaneously through zero stress, whence the conclusion:

The least material is required in a framework of given figure to resist a given loading with prescribed intensities of stress in its members, for some distribution of stress which makes zero the stress in as many bars as are superfluous.

That is:

A statically determined framework of included figure is the most economic form of a framework of given indeterminate figure for the support of a given loading.

A simple and much more objective proof of this same fact, on the same basis, due to Professor George F. Swain, will be found in the article in the *Technology Quarterly*, before mentioned. But, it may be objected that both these proofs are based on the simple rule of design $A = \frac{S}{\sigma}$, whereas posts are usually designed by more complicated formulas. Now, it can easily be shown that the change in volume of a given framework due to a slight change $d S_i$ in the stress in one of the superfluous bars i of the framework, independently of all others, is expressed by

$$d V = d S_i \sum_{f=1}^{f=m} \frac{S'_f(i)\, l_f}{\phi'_f (A_f)}$$

where $S_f = \phi_f (A_f)$ expresses the relation between the section area of a bar and its stress and ϕ' is the rate of change of ϕ. Now, providing that $\phi'_f (A_f)$ in no case diminishes numerically as A increases, that is, providing that a post never gains less in strength for a given increase in its area as the area increases, $d V$ will not change its sign from negative to positive during continuous variation, and therefore changing S_i so as to make $d V$ negative, that is, so as to diminish the total volume of material employed, will continue to diminish that volume until a point of discontinuity is attained where some bar's section is reduced to zero. As before, we reach the conclusion that as many bars as are superfluous must have their sections reduced to zero for most economic proportions. But the condition as to post formula on which this conclusion is founded is true of all formulas of the Gordon-Rankine type, those almost universal in practice; therefore, the conclusion as to the superior economy of an included statically determined design holds, under the present standard rules of design.

We may note a further consequence, relating to stiffness. Any change in the positions of the joints, therefore the deflection of any framework, is completely defined by the strains of the bars left after excluding in any way as many bars as are superfluous (see page 365). That is, the deflection of any indeterminate framework from a state of no stress under no load (the common supposition), to its actual state of stress under a given load, is precisely the same as that of any of the determined frameworks of included figure, which we may design to carry the same loading with the same intensities of stress in the corresponding bars. But at least one of these latter frameworks would

carry this load with less material in its bars (see page 368), and therefore be stiffer construction, since, with the same amount of material in its bars, its deflections would be smaller.

Thus we see that a given loading can always be carried, with precisely the same deflections, with the same intensities of stress and with less material in the bars, by a statically determined framework the figure of which is included in that of the given indeterminate framework, than by the given framework, which may be any whatever. These same considerations, by simple modification, yield the same result as applied to the cost of the material where different materials are used, as steel cable for tension members and rolled metal for compression members.

What of the bearing of these facts? While it is true that they hold strictly only for the case of a single loading, their weight for all cases is very considerable.

"Frequently, perhaps usually in practice, some one loading or combination of loadings determines the dimensions of almost all the principal members of the framework, the members not thus determined affecting the total quantity of material employed but slightly. This is particularly the case for large structures in which dead weight plays a leading part, and where the total quantity of material employed is so large that economy in this direction is particularly desirable. In such cases one loading virtually rules the economic design, and our foregoing results hold approximately true. *The consideration of the effects of changes in loading in these cases*, although of great importance as far as the strength of the structure goes, *is of wholly secondary significance from our economic point of view.*"

With this quotation from the article in the *Technology Quarterly*, where all the foregoing will be found more fully explained as well as illustrated, we will turn to the more general and new part of the subject, the exact design of indeterminate frameworks subject to several different loadings.*

Perhaps the most readily comprehensible derivation and most simple form of the equations whose solution furnishes an exact design of any indeterminate framework of given figure subject to any given series of loadings and under any given conditions of design, is the following: As we have already noted on page 366, if, under any loading, 1, the

* The writer here desires to state his indebtedness to Professor George F. Swain for the suggestion of the possibility of such exact design, and, further, to acknowledge many other valuable suggestions, as well as a critical review of the present article, by Professor Swain, under whom the writer first studied structures and learned to appreciate the superiority of statically determined construction.

stresses in the members be denoted by S_{f_1} then the unstrained length of any superfluous bar i is shorter than the length of its line in the given figure of the framework by

$$\sum_{f=1}^{f=m} \frac{S_{f_1} \; S'_{f^{(i)}} \, l_f}{E_f \, A_f}$$

And if under any other loading, 2, the stresses in the same framework are S_{f_2} then the same quantity is given by

$$\sum_{f=1}^{f=m} \frac{S_{f_2} \; S'_{f^{(i)}} \, l_f}{E_f \, A_f}.$$

But since the amount by which the unstrained length of any superfluous bar is shorter than the length of its line in the given figure of the framework is not, by supposition, subject to change, through change of loadings (or rather, of the stresses existing under them), we will have, for each superfluous bar i, an equation of the form

$$\sum_{f=1}^{f=m} \frac{S_{f_1} \; S'_{f^{(i)}} \, l_f}{E_f \, A_f} = \sum_{f=1}^{f=m} \frac{S_{f_2} \; S'_{f^{(i)}} \, l_f}{E_f \, A_f}$$

or, in briefer form,

$$\sum_{f=1}^{f=m} \frac{(S_{f_1} - S_{f_2}) \; S'_{f^{(i)}} \, l_f}{E_f \, A_f} = 0$$

Now, whatever our rules of design may be, the section areas A_f will be definite functions, of some sort, of the stresses S_f; and the stresses S_f are in turn linear functions of the stresses S_i in the superfluous bars through the equations

$$S_{f_1} = S_{f_{0_1}} + \sum_{i=h}^{i=m} S_{i_1} \; S'_{f^{(i)}} \qquad S_{f_2} = S_{f_{0_2}} + \sum_{i=h}^{i=m} S_{i_2} \; S'_{f^{(i)}}, \text{ etc.},$$

where the $S_{f_{0_1}}$ are the stresses that load 1 would cause in the statically determined framework composed of bars 1 to g and the $S_{f_{0_2}}$ are the corresponding stresses which load 2 would cause in that framework. Thus the equations shown are implicit equations of necessary relations between the stresses S_{i_1}, S_{i_2}, etc., in the superfluous bars i under the various loadings 1, 2, etc. For each superfluous bar i and for each pair of loadings there is one such equation, that is, if there be N different loadings, for each bar i there will be $N-1$ such

independent equations,* and if there be k superfluous bars in all, this will give a total of $k\ (N-1)$ such independent equations between the superfluous bar stresses S_{i_1}, S_{i_2}, etc. But of these latter there are N for each bar i, that is, the stress in the bar i under each one of the N different loadings, and therefore in all there are $k\,N$ of the superfluous bar stresses S_{i_1}, S_{i_2}, etc., between which the above $k\ (N-1)$ necessary relations subsist. It follows that the values of the S_i (and therefore the design itself), possess a k^{tuple} degree of indetermination, that is, in general, k of the S_i may be arbitrarily assumed, or the stress in each superfluous bar under one loading may be arbitrarily taken, or, more generally, any k further consistent relations between the stresses S_i may arbitrarily be set, and then a solution rigorously satisfying the given conditions of design as expressed through the primary equations, directly obtained. For, the solution of the equations under these circumstances furnishes us the stresses S_i in the superfluous bars under all the given loadings, thence at once all the stresses under the given loadings, thence through the prescribed conditions of design all the section areas, and finally thence, as previously explained, the precise unstrained lengths of the superfluous bars. But here a criticism is necessary. The complete mathematical solution of such a set of equations will, in general, consist of a considerable number of separate solutions, not all of which may be real, and of the real solutions, some or all of which may be inconsistent with conditions of design not expressible in the equations. These will generally be conditions of inequality such as that in the design thus obtained the most unfavorable demands shall be those supposed most unfavorable in drawing up the equations, and therefore the basis of those equations. For example, the rule of design being that each bar under its most unfavorable loading shall have stress of a prescribed intensity, in drawing up the equations, the most unfavorable loading for each bar must, in advance, be judged, the equations drawn up accordingly, and then the solutions thus obtained, which are real (and which will necessarily satisfy the condition that under the selected loading for each bar it has the prescribed intensity of stress), must further be tested as to their satisfying the suppositions as to most unfavorable loadings under which the equa-

* If two extremes of temperature were also to be considered, the effect would be to double the number of loadings. Temperature may be taken into consideration by putting $\frac{S_f}{E_f A_f} + \tau_f t_f$ for $\frac{S_f}{E_f A_f}$ where τ_f is the coefficient of expansion.

tions were drawn up. Frequently, none of the real mathematical solutions, where these exist, will satisfy these conditions, and thus limitations arise as to the arbitrary setting of stresses in the superfluous bars, or more generally, the arbitrary setting of k additional relations between these stresses. Nevertheless, a k^{tuple} infinity of designs of an indeterminate framework of given figure, under given loadings and subject to given conditions of design, is in general possible. It remains for us first to note some of the consequences, and second, to illustrate them by simple examples.

One will be most impressed, perhaps, by the fact of the possible multiplicity of exact and admissible designs of frameworks of given figure subject to given loadings. Clearly, any one of these designs will not necessarily be a good design, and, therefore, the fact that a design for an indeterminate framework of given figure subject to given loadings, rigorously fulfills the prescribed conditions of design is no indication whatsoever that the design in question is to be approved. Some such designs will be far more economic than others, that is, will require far less material; some such designs will be far stiffer than others, or will have other points of superiority. Our end is not simply to obtain any such design, but the best of such designs.

But what is the best such design? Suppose we say that design which is most economic of material is the best. The material in the bars is expressed by

$$\sum_{f=}^{f=m} A_f l_f$$

which is a function, through the prescribed conditions of design, of the bar stresses S_f, that is, finally, of the superfluous bar stresses S_i. We may determine mathematically the necessary relations for this to be a minimum, make these the additional arbitrary relations and thus obtain the most economic solutions. The most economic of these economic designs will be that sought. Theoretically the way is clear. And what will be the nature of this most economic design? No longer, as in the case of design under a single loading, will it necessarily be some statically determined framework of included figure. The fact that the volume of material no longer is expressible on any basis of design as a linear function of independent variables would alone

suggest this, and illustrations may easily be found which conclusively prove it. But, as will later be shown, this possible superiority of an indeterminate design, which will only occur when a structure is subject to very diverse principal loadings, and not at all, or in but small degree, to large structures, the dead weight of which is prominent, is never likely to be considerable, and only exists at all through the neglect of temperature stresses to which the indeterminate structure is subject where the determinate structure is not, and the further neglect of all consequences of dimensioning and work in erection of other than the most extreme accuracy, the neglect of any lack of absolute resistance of all supports, and the neglect of any yielding at the joints and set or other yielding under operation; all of which must in practice exist, and considerably affect the indeterminate construction where they inappreciably affect the determinate. Finally, simply as an interesting although perhaps not practically important fact, it may be observed that by a combination of statically determined designs, of figures included within that of a given indeterminate framework, or what the writer calls a multiple design, a series of different loadings can always be carried more economically with determinate stresses than by even the most economic of single indeterminate designs of the given figure, so that the conclusion is absolutely warranted that:

"Statical indetermination in a structure is always to be regarded as self-interference with efficiency."

Turning to the question of stiffness, but little can be added to what has already been said. Indeterminate designs subject to several loadings will in general have deflections of a different character from those of any one of the determinate designs of included figure. It is, therefore, impossible to set any basis of comparison that would not at once be subject to much criticism. The writer will merely observe that the comparisons between such structures, with which he is familiar, have usually been made on most unfair bases, and some too generally accepted conclusions, notably on the comparative stiffness of three-hinged and two-hinged or hingeless arches, are not justifiable. The writer's own very limited but fairly equable comparisons show clearly that no very marked superiority can be attributed to either type in this respect, and that points of superiority in one are offset by other

points of superiority in the other, so that nothing but a debatable conclusion in any case is possible. Some later illustrations will make this clear. And in fairness it must be conceded that the fact that in certain cases, in the attempt to secure static determination through the use of certain devices, play has been introduced, most objectionable in connection with loads moving at high speed, only shows the defective character of the devices used, and in no wise attains the principle the application of which does not necessarily involve the use of these devices.

Lastly, what of the superior safety of frameworks with superfluous bars? Since there are superfluous bars there are bars the rupture of which does not necessarily involve the failure of the framework, and this is certainly an element of safety; but, generally, not every bar may be regarded as a superfluous bar, so that in an indeterminate framework the rupture of some bars would not less necessarily involve the failure of the structure than in the case of a determinate framework. Finally, even of the bars the omission of which would leave the remaining portion statically determined, and, therefore, which, from this point of view, are not essential, some, perhaps many, will be of such actual importance in the structure that their removal would result in serious overstrain of some of the remaining bars, and thereby failure; while of those still remaining the sudden rupture of some and the accompanying shock would be fatal to the structure. The superior safety due to the use of superfluous bars is evidently very limited; and, when we note that the statically determined structures require less material to satisfy the same conditions of design, it becomes questionable whether for equal amounts of material the determinate design, which would then have lower unit stress than the indeterminate design, would not, therefore, be at least as safe.

Turning from these general considerations let us examine a little more closely the equations through the solution of which exact designs are to be obtained. The implicit and symbolic form in which we have stated them gives them a most deceptive appearance of simplicity which is very far from corresponding with facts. In explicit form we should find each equation to consist of a series of terms in the form of fractions, each fraction having for numerator a linear function of the variables S_i, but for denominator a linear or

higher degree function of the variables, according to the complexity of the rules of design used, and, very likely, it would not be a whole integral function. The simplest possible rule of design, making the section areas simply proportional to the greatest stress borne—which is the common rule for tension members—makes the denominator linear in terms of the variables S_i. But the introduction of post formulas of the Gordon-Rankine type, now generally used, at once would introduce fractional functions of the variable into the denominator. Now, these equations in fractions have a most delusive appearance of simplicity. Only when we have accomplished their reduction to whole integral equations, a very considerable although theoretically simple undertaking in itself, are we able to form some conception of what is involved in obtaining a solution. Then we discover that we have a series of simultaneous equations of higher degree than the first between numerous unknowns. The indetermination having been removed by the introduction of suitable arbitrary relations, by perfectly well-known methods of elimination, these equations are eventually reducible to a single equation with one unknown, the solution of which then by numerical methods (for, of course, we are dealing with equations with known numerical coefficients), offers no theoretic difficulty; but practically in any actual case the work involved would be appalling. And then the solutions obtained may prove worthless, as they may not be consistent with the suppositions on which the equations were drawn up. The number of such possible suppositions will generally be very considerable, and it may well happen that many trials will be necessary before admissible solutions will be found, thus multiplying the labor many times; and, if we wished to be complete in our mathematical solution, all the possible suppositions should be examined, making an undertaking from which even the bravest calculator might well recoil.

In order to give a conception, even though faint, of the appearance of one of the primary equations when written explicitly, but simply and in fractional form, such an equation for the bar i, when there are only the two different loadings 1 and 2, and when the basis of design is simply $A = \frac{S}{\sigma}$, is given, as follows:

$$\sum_{b=1}^{b=c} \frac{\sigma_b (S_{b_{o_1}} - S_{b_{o_2}}) S'^{(i)}_b l_b}{E_b(S_{b_{o_1}} + \sum_{i=h}^{i=m} S_{i_1} S'^{(i)}_b)} + \sum_{e=d}^{e=g} \frac{\sigma_e (S_{e_{o_1}} - S_{e_{o_2}}) S'^{(i)}_e l_e}{E_e(S_{e_{o_2}} + \sum_{i=h}^{i=m} S_{i_2} S'^{(i)}_e)}$$

$$+ (S_{h_1} - S_{h_2}) \left[\sum_{b=1}^{b=c} \frac{\sigma_b S'^{(h)}_b S'^{(i)}_b l_b}{E_b (S_{b_{o_1}} + \sum_{i=h}^{i=m} S_{i_1} S'^{(i)}_b)} + \sum_{e=d}^{e=g} \frac{\sigma_e S'^{(h)}_e S'^{(i)}_e l_e}{E_e (S_{e_{o_2}} + \sum_{i=h}^{i=m} S_{i2} S'^{(i)}_e)} \right]$$

$$+ (S_{(h+1)_1} - S_{(h+1)_2}) \left[\sum_{b=1}^{b=c} \frac{\sigma_b S'^{(h+1)}_b S'^{(i)}_b l_b}{E_b (S_{b_{o_1}} + \sum_{i=h}^{i=m} S_{i_1} S'^{(i)}_b)} + \sum_{e=d}^{e=g} \frac{\sigma_e S'^{(h+1)}_e S'^{(i)}_e l_e)}{E_e (S_{e_{o_2}} + \sum_{i=h}^{i=m} S_{i_2} S'^{(i)}_e)} \right]$$

$$+ \cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots$$

$$+ (S_{i_1} - S_{i_2}) \left[\sum_{b=1}^{b=c} \frac{\sigma_b (S'^{(i)}_b)^2 l_b}{E_b (S_{b_{o_1}} + \sum_{i=h}^{i=m} S_{i_1} S'^{(i)}_b)} + \sum_{e=d}^{e=g} \frac{\sigma_e (S'^{(i)}_e)^2 l_e}{E_e (S_{e_{o_2}} + \sum_{i=h}^{i=m} S_{i_2} S'^{(i)}_e)} + \frac{\sigma_i l_i}{E_i S_{i_1}} \right]$$

$$+ \cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots$$

$$+ (S_{m_1} - S_{m_2}) \left[\sum_{b=1}^{b=c} \frac{\sigma_b S'^{(m)}_b S'^{(i)}_b l_b}{E_b (S_{b_{o_1}} + \sum_{i=h}^{i=m} S_{i_1} S'^{(i)}_b)} + \sum_{e=d}^{e=g} \frac{\sigma_e S'^{(m)}_e S'^{(i)}_e l_e}{E_e (S_{e_{o_2}} + \sum_{i=h}^{i=m} S_{i_2} S'^{(i)}_e)} \right]$$

$$= 0.$$

Here, it is supposed that bars b, ($b = 1$ to c), and the particular superfluous bar i for which the equation is written, have their greatest stresses under the loading 1, while the bars e, ($e = d$ to g), have their greatest stresses under loading 2. This equation is still in abbreviated and symbolic form, being only written in part and shortened by the use of the symbol Σ. Remember that for each remaining of the k superfluous bars there is a similar equation. These equations cleared of fractions would each be of the $(m - k + 1)$ degree in the S_i, that is, of degree one higher than the number of bars in the framework not superfluous. Had there been N different loadings instead of but two there would have been not simply k such equations but $k(N - 1)$ such equations, as previously pointed out. But this is not all. These equations were drawn up on the supposition that certain bars had their greatest stresses under loading 1, the others their greatest stresses under loading 2. We may or may not find solutions consistent with this, and, in any case, to be complete we should investigate all other possible cases which are 2^m in number, that is, we should investigate, perhaps solve, 2^m such sets of k equations. But had we N loadings instead of only two, this number of possible

cases for investigation would have been further and greatly increased. So it will be seen that a complete, exact, mathematical solution, even on the simplest possible conditions of design, involves great labor, very rapidly increasing as the number of bars in the framework increases. The justness of the statement, made earlier, that the exact method was not to be thought of for use in practical designing, will now be perceived, and it is thought that the illustrations following will amply confirm this. But since exact methods are not applicable in practice, and the tentative methods of the past will continue to be used, and at best can never give better than most imperfect results, the impossibility of benefiting through the use of indeterminate designs, of even their theoretically possible advantages, will be clear; a final and fatal fact for the adherents to such construction. If ever an ideal has been completely vindicated by increased knowledge it is certainly that of the statically determined construction of frameworks.

Illustrations of Design of Indeterminate Frameworks under Multiple Loadings.

Let it be required to design a framework of the form and dimensions shown in Fig. A, so that it shall support either

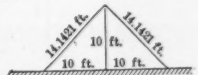
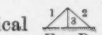


Fig. A.

the pull P in a horizontal direction, thus Fig. B. or the weight W, thus Fig. C. so that each bar, under its most unfavorable load, shall have stress of the mean intensity of 5 tons per square inch. Denoting the diagonals by the numbers 1 and 2, respectively, and the vertical  Fig. D. by the number 3 (see Fig. D), and calling the loading with the horizontal pull the loading P, and the loading with the vertical weight the loading W, let us suppose that in the bars 1 and 2 the maximum stress occurs under the loading P and in the bar 3 under the loading W, so that

$$A_1 = \frac{S_{1_P}}{+5} \qquad A_2 = \frac{S_{2_P}}{-5} \qquad A_3 = \frac{S_{3_W}}{-5}$$

the bar 1 being designed under tensile stress and the bars 2 and 3 under compression.

Treat bar 3 as the superfluous bar. Then unity tension in it would cause compression of 0.707 in each of the bars 1 and 2 or $S'_1 = S'_2 = -0.707$ (see Fig. E). The apex of the triangle sinks more under the load W than under the pull P by

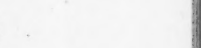


FIG. E.

$$\frac{S'_1 (S_{1_W} - S_{1_P}) l_1}{EA_1} + \frac{S'_2 (S_{2_W} - S_{2_P}) l_2}{EA_2} = (S_{3_P} - S_{3_W}) \frac{l_3}{EA_3}$$

and, noting the values of S', l and A, this gives the simple relation

$$\frac{S_{1_P} - S_{1_W}}{S_{1_P}} + \frac{S_{2_P} - S_{2_W}}{-S_{2_P}} = \frac{S_{3_P} - S_{3_W}}{-S_{3_W}}$$

$$\text{Now, } S_{1_P} = S_{1_{o_P}} + S'_1 S_{3_P} = +0.707P - 0.707S_{3_P}$$

$$S_{1_W} = S_{1_{o_W}} + S'_1 S_{3_W} = -0.707W - 0.707S_{3_W}$$

$$S_{2_P} = S_{2_{o_P}} + S'_2 S_{3_P} = -0.707P - 0.707S_{3_P}$$

$$S_{2_W} = S_{2_{o_W}} + S'_2 S_{3_W} = -0.707W - 0.707S_{3_W}$$

so that the relation may be written

$$\frac{P + W - S_{3_P} + S_{3_W}}{P - S_{3_P}} - \frac{P - W + S_{3_P} - S_{3_W}}{P + S_{3_P}} = \frac{S_{3_W} - S_{3_P}}{S_{3_W}}$$

or, clearing of fractions,

$$2PS^2_{3_W} + (2PW - P^2 + S^2_{3_P}) S_{3_W} + P^2 S_{3_P} - S^3_{3_P} = 0$$

which is a quadratic equation in S_{3_W} and a cubic in S_{3_P}. We observe the following relations:

If $S_{3_W} = 0$, then $S_{3_P} = 0$, the bar 3 having zero area.

If $S_{3_P} = \pm P$, then $S_{3_W} = -W$, either bar 1 or bar 2 having zero area.

If $S_{3_P} = 0$, then $S_{3_W} = -W + \frac{P}{2}$, bars 1 and 2 being of the same area and bearing together $\frac{P}{2}$, whereby their junction sinks the full allowable strain of 3.

We note that S_{3_P} is the primary stress in bar 3, since it does not vary with P, and the primary stresses in 1 and 2 are $-0.707\, S_{3_P}$.

It will be seen that the bar 3 can only be used to advantage when W is greater than P, which we will suppose to be the case.

Since 1 and 2 are most efficient in resisting P we have supposed them designed under P, while 3, which is only efficient in resisting W,

we have supposed designed under W. Thus we must satisfy the inequalities

$$S^2_{1_P} \geqq S^2_{1_W} \qquad S^2_{1_P} \geqq \frac{S^2_{3_P}}{2}$$

$$S^2_{3_W} \geqq S^2_{3_P} \text{ and also}$$

$$S^2_{2_P} \geqq S^2_{2_W} \qquad S^2_{2_P} \geqq \frac{S^2_{3_P}}{2}$$

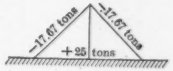
The latter pair of inequalities are necessitated by the primary stresses.

A consequence of this last pair of inequalities is that $S^2_{3_P} \leqq \left(\frac{P}{2}\right)^2$ so that $\frac{(S_{1_P} - S_{2_P})\, l_1}{5}$ is the volume of material in the bars 1 and 2 under these circumstances, and is constant. For economy then, we must so choose S_{3_P} within the above limits that, while satisfying the remaining inequalities, $S^2_{3_W}$ shall be as small as possible.

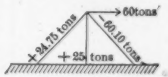
Let us take $W = 100$ tons, $P = 60$ tons, and assume $S_{3_P} = 25$ tons. We have $S^2_{3_W} + \frac{9025}{120} S_{3_W} + (750 - \frac{15625}{120}) = 0$, whence $S_{3_W} = -37.6 \pm 28.2 = -9.4$ tons, or -65.8 tons, of which the former is excluded by the condition $S^2_{3_W} \geqq S^2_{3_P}$.

Taking the solution of $S_{3_W} = -65.8$ tons, we obtain the following:

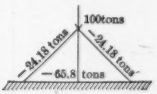
With no load, $S_1 = S_2 = -17.67$ tons, $S_3 = +25$ tons.



With load $P = 60$ tons, $S_{1_P} = +24.75$ tons, $S_{2_P} = -60.10$ tons, $S_{3_P} = +25$ tons.



With load $W = 100$ tons, $S_{1_W} = -24.18$ tons, $S_{2_W} = -24.18$ tons, $S_{3_W} = -65.8$ tons.



The section areas are: (bar 1) $\frac{24.75}{5} = 4.95$ sq. ins., (bar 2) $\frac{-60.10}{-5} = 12.02$ sq. ins., and (bar 3) $\frac{-65.8}{-5} = 13.16$ sq. ins.

The volumes are: (bar 1) 840.0 cu. ins., (bar 2) 2 039.8 cu. ins., (bar 3) 1 579.2 cu. ins., a total of 4 459 cu. ins.

Consider the following comparisons:

The statically determined form has areas (bar 1) 16.97 sq. ins., (bar 3) 20 sq. ins.; and volumes (bar 1) 2 879.8 cu. ins., (bar 3) 2 400 cu. ins., a total of 5 279.8 cu. ins.

The statically determined form has areas (bars 1 and 2) each 14.14 sq. ins.; and volumes (bars 1 and 2) each 2 400 cu. ins., a total of 4 800 cu. ins.

But the combination or "multiple" form for P and for $W - P$ has areas (bars 1 and 2) each 8.48 sq. ins., and (bar 3) 8 sq. ins.; and volumes (bars 1 and 2) each 1 440 cu. ins., (bar 3) 960 cu. ins., a total of but 3 840 cu. ins.

This illustrates clearly how the indeterminate form may be more economic than any single included determinate form, yet not as economic as a combination of determinate forms.

The second value of S (— 9.4 tons), excluded by the condition $S^2_{3P} \gtreqless S^2_{3W}$, appears in the solution correctly as one of the answers to the unrestricted problem. This partial exclusion of solutions, as previously noted, would usually occur in problems of this description, and often not even one of the solutions would be admissible under the limitations prescribed.

Thus, in this case, S_{1P} being tension and S_{1W} being compression, the condition $S^2_{1P} \gtreqless S^2_{1W}$ may be written $S_{1P} \gtreqless - S_{1W}$; whence $P - S_{3P} \lesseqgtr W + S_{3W}$. The limiting value of this relation is $P - W - S_{3P} = S_{3W}$, or, in this case, $40 + S_{3P} = - S_{3W}$. If we substitute this in the equation

$$\frac{P + W - S_{3P} + S_{3W}}{P - S_{3P}} - \frac{P - W + S_{3P} - S_{3W}}{P + S_{3P}} + \frac{S_{3P} - S_{3W}}{S_{3W}} = 0$$

this becomes $2 - \frac{2 S_{3P}}{60 + S_{3P}} - \frac{40 + 2S_{3P}}{40 + S_{3P}} = 0$; or $S^2_{3P} + 20S_{3P} - 1\,200 = 0$, a quadratic, of which $S_{3P} = + 26.056$ is one root, and the corresponding value of S_{3W} is $- (40 + S_{3P}) = - 66.05$. If we were to assume S_{3P} greater than $+ 26.056$, there would be no admissible solution.

At first glance it would seem that the greater S_{3P}, the smaller (numerically) would be S_{3W}; but if we differentiate the equation of

relation between S_{3P} and S_{3W} and put $\frac{d S_{3W}}{d S_{3P}} = 0$, we obtain the condition

$$S_{3W} = \frac{3}{2}\left(\frac{S_{3P}^2 - \frac{1}{3}P^2}{S_{3P}}\right) \text{ or, in this case, } S_{3W} = \frac{3}{2}\left(\frac{S_{3P}^2 - 1\,200}{S_{3P}}\right),$$

and substituting this in the equation of relation between S_{3P} and S_{3W}, we obtain the quintic

$$S_{3P}^6 + 540 S_{3P}^4 + 28\,800 S_{3P}^3 - 1\,296\,000 S_{3P}^2 - 30\,240\,000 S_{3P} + 777\,600\,000 = 0.$$

One root is $S_{3P} = +19.168$, and the corresponding (minimum) value of S_{3W} is $S_{3W} = -65.14$, which gives only slightly more economy than $S_{3P} = +25$.

Thus far we have examined only the case of bars 1 and 2 with maximum loads under P, and bar 3 with maximum load under W, and, moreover, the suppositions $S_{1P} > 0$, $S_{2P} < 0$, $S_{3W} < 0$. These latter were not, perhaps, necessary, but apparently are essential to economic solutions. On the other hand, although the argument that the bars 1 and 2 should have their maximum loads under P for a most economic design looks reasonable, it may be misleading, so let us examine some further cases.

As a second case suppose bar 1 to have its maximum stress under W (instead of under P), bar 2 having its maximum stress under P, and bar 3 its maximum stress under W as before. Here we will suppose $S_{1W} < 0$, $S_{2P} < 0$ and $S_{3W} < 0$, so that the bars are all designed under compression. The equation is

$$\frac{S_{1P} - S_{1W}}{-S_{1W}} + \frac{S_{2P} - S_{2W}}{-S_{2P}} = \frac{S_{3P} - S_{3W}}{-S_{3W}}$$

or, replacing S_{1P}, S_{1W}, S_{2P} and S_{2W} by their values in P, W, S_{3P} and S_{3W},

$$\frac{P + W - S_{3P} + S_{3W}}{W + S_{3W}} - \frac{P - W + S_{3P} - S_{3W}}{P + S_{3P}} + \frac{S_{3P} - S_{3W}}{S_{3W}} = 0,$$

which, cleared of fractions, gives

$$W S_{3P}^2 + (P\,W + P\,S_{3W} - W\,S_{3W} - S_{3W}^2)\,S_{3P} + (P^2 - P\,W + W^2)\,S_{3W} - (P - 2\,W)\,S_{3W}^2 + S_{3W}^3 = 0,$$

which is a quadratic in S_{3P} and a cubic in S_{3W}. Now, we have in this case

$$A_1 = \frac{S_{1W}}{-5} = \frac{0.707\,(W + S_{3W})}{5} \text{ sq. ins.} \qquad l_1 = 12 \times 14.14 \text{ ins.}$$

$$A_2 = \frac{S_{2P}}{-5} = \frac{0.707\,(P + S_{3P})}{5} \text{ sq. ins.} \qquad l_2 = 12 \times 14.14 \text{ ins.}$$

$$A_3 = \frac{S_{3W}}{-5} = \frac{-S_{3W}}{5} \text{ sq. ins.} \qquad l_3 = 12 \times 10 \text{ ins.}$$

$$\text{Total volume} = A_1 l_1 + A_2 l_2 + A_3 l_3 = 24\,(P + W + S_{3P}).$$

It is clear that for economy we must aim to make S_{3P} as small as possible while satisfying the inequalities $S^2_{1W} > S^2_{1P}$, $S^2_{2P} > S^2_{2W}$, $S^2_{3W} > S^2_{3P}$. The first two of these conditions combined require $S^2_{2P} > S^2_{1P}$, since $S_{1W} = S_{2W}$ necessarily; and thence follows that $S_{3P} > 0$. Since $S_{2P} \leqq 0$, $-S_{2P} \geqq -S_{2W}$, or $P - W + S_{3P} - S_{3W} \geqq 0$.

If $P + S_{3P} = W + S_{3W}$, then $\dfrac{P + W - S_{3P} + S_{3W}}{W + S_{3W}} + \dfrac{S_{3P} - S_{3W}}{S_{3W}} = 0$;

whence $S_{3P} = P\ \dfrac{W - P}{W + P} = +15$ tons, $S_{3W} = -25$ tons.

Now, $S_{3P} - S_{3W}$ diminishes with S_{3P} at these values, and they alone satisfy $P + S_{3P} - W - S_{3W} = 0$. Thus, if $P + S_{3P} - W - S_3 \geqq 0$, then $S_{3P} \geqq P\ \dfrac{W - P}{W + P}$. At the limit we may take $S_{3P} = P\ \dfrac{W - P}{W + P}$ $= 15$ tons, and this is the least allowable value of S_{3P} in this case. The corresponding values are:

$S_{3W} = P - W + S_{3P} = -25$ tons,

$S_{2P} = S_{2W} = S_{1W} = -0.707\,(P + S_{3P}) = -53.02$ tons,

$S_{1P} = 0.707\,(P - S_{3P}) = +31.81$ tons,

$A_1 = A_2 = 10.60$ sq. ins.,

$A_3 = 5$ sq. ins.,

Vol. bar 1 = vol. bar 2 = 1 800 cu. ins.,

Vol. bar 3 = 600 cu. ins.

Total volume of bars = 4 200 cu. ins., which is a considerably more economic design than that of our first case. We note that, since $A_1 = A_2$ $S_{3P} = +15$ tons is also the primary stress in bar 3.

As a third case, suppose bar 1 has its maximum stress under P, but bar 2 has its maximum stress under W, and bar 3 has its maximum stress under W as before. In this case $S^2_{1P} > S^2_{1W}$, $S^2_{2W} > S^2_{2P}$, $S^2_{3W} > S^2_{3P}$.

Since $S_{1W} = S_{2W}$, $S^2_{1P} > S^2_{2P}$; whence $S_{3P} < 0$, $S_{1P} > 0$ and $S_{3W} < 0$. This latter follows from $S_{1P} > -S_{1W}$, or $P - S_{3P} > W + S_{3W}$ and $S^2_{3W} > S^2_{3P}$.

Now, if we suppose $S_{2W} = S_{1W} < 0$, then the equation would be

$$\frac{S_{1P} - S_{1W}}{S_{1P}} + \frac{S_{2P} - S_{2W}}{-S_{2W}} = \frac{S_{3P} - S_{3W}}{-S_{3W}}$$

which we may write $1 - \frac{S_{1W}}{S_{1P}} - \frac{S_{2P}}{S_{2W}} + \frac{S_{3P}}{S_{3W}} = 0$. Since the terms following the first are numerically less than unity by supposition, this equation asserts that $1 + (>0) \pm (<1) + (>0) = 0$, which is impossible.

If we suppose $S_{2W} = S_{1W} > 0$, then the equation would be

$$\frac{S_{1P} - S_{1W}}{S_{1P}} + \frac{S_{2P} - S_{2W}}{S_{2W}} = \frac{S_{3P} - S_{3W}}{-S_{3W}}$$

which we may write $-1 - \frac{S_{1W}}{S_{1P}} + \frac{S_{2P}}{S_{2W}} + \frac{S_{3P}}{S_{3W}} = 0$

or $-1 - (>0) \pm \left(\begin{matrix}<1\\>0\end{matrix}\right) + (<1) = 0$, which is possible only if $S_{2P} > 0$, that is, if $P + S_3 < 0$, or if $S_{3P} < -P$. Since $S_{2W} > 0$, $-(W + S_{3W}) > 0$, or $S_{3W} < -W$. Now, $A_1 = \frac{0.707\,(P - S_{3P})}{5}$, $A_2 = \frac{-0.707\,(W + S_{3W})}{5}$, $A_3 = \frac{-S_{3W}}{5}$, $A_1 l_1 + A_2 l_2 + A_3 l_3 = 24\,(P - W - S_{3P} - 2\,S_{3W})$ cu. ins. This will be a minimum for $S_{3P} + 2\,S_{3W}$ a maximum. But the maximum values $S_{3P} = -P$ and $S_{3W} = -W$ occur simultaneously, making $S_{3P} + 2\,S_{3W} = -(P + 2\,W)$ a maximum, therefore, we have

$S_{3P} = -60$ tons,	$A_1 = 16.96$ sq. ins.,	Vol. bar $1 = 2\,880$ cu. ins.,
$S_{3W} = -100$ tons,	$A_2 = 0$,	Vol bar $2 = 0$,
$S_{1P} = +84.84$ tons,	$A_3 = 20$ sq. ins.,	Vol. bar $3 = 2\,400$ cu. ins.
$S_{2P} = S_{1W} = S_{2W} = 0$,		

Total volume, $= 24\,(60 - 100 + 60 + 200) = 5\,280$ cu. ins.

This "economic" solution is a statically determined form.

As a fourth case let all the bars have their maximum stresses under W, then clearly $S_{1W} < 0$, $S_{2W} < 0$, and the equation is

$$\frac{S_{1P} - S_{1W}}{-S_{1W}} + \frac{S_{2P} - S_{2W}}{-S_{2W}} = \frac{S_{3P} - S_{3W}}{-S_{3W}}$$

Here $A_1 = A_2$, and consequently S_{3_P} is the primary stress in 3. $A_1 = \frac{S_{1_W}}{-5} = \frac{0.707\,(W + S_{3_W})}{5} = A_2$, $A_3 = \frac{-S_{3_W}}{5}$, $A_1 l_1 + A_2 l_2 + A_3 l_3 =$ 24 $(2\,W + S_{3_W})$ cu. ins. This will be a minimum for S_{3_W} a minimum. Now, from $S^2_{2_W} > S^2_{2_P}$ follows $W + S_{3_W} > P + S_{3_P}$, or $S_{3_W} > P - W + S_{3_P}$, or $W + S_{3_W} - S_{3_P} > P$. But $\frac{S_{1_P} - S_{1_W}}{-S_{1_W}} + \frac{S_{2_P} - S_{2_W}}{-S_{2_W}} + \frac{S_{3_P} - S_{3_W}}{+S_{3_W}} = 0$ gives $(W + S_{3_W} - S_{3_P})\,S_{3_W} + W\,S_{3_P} = 0$; whence $P\,S_{3_W} + W\,S_{3_P} > 0$, since $S_{3_W} < 0$. This gives $S_{3_P} > -\frac{P}{W} S_{3_W}$; whence $S_{3_W} > P - W - \frac{P}{W} S_{3_W}$, or $S_{3_W} > W\frac{P - W}{P + W}$, or $S_{3_W} > -25$. At limit $S_{3_W} = -25$ tons (minimum), $S_{3_P} = +15$ tons, $S_{1_P} = +31.81$ tons, $S_{2_P} = S_{2_W} = S_{1_W} = -53.02$ tons; $A_1 = A_2 = 10.60$ sq. ins., $A_3 = 5$ sq. ins. Total volume $= 24\,(200 - 25) = 4\,200$ cu. ins., which is identical with the most economic design of the second case.

We might consider further cases in which bar 3 was designed under the load P, but it seems sufficiently evident that these could not be economic designs and need no study.

The final conclusion is that in the most economic design all bars have their maximum stresses under the vertical load, and one of the bars has the same stress under the horizontal pull, a result somewhat different from that anticipated at the outset.

As a last study, suppose it were required that the frame should support any one of the three loadings , then solutions of the first three of these preceding cases would not in general apply, since they are non-symmetrical. But solutions of the last of the cases preceding evidently would apply, since they are symmetrical, and the most economic solution will be the same here as there. That is to say, the design with $A_1 = A_2 = 10.60$ sq. ins., $A_3 = 5$ sq. ins., and having a volume of 4 200 cu. ins., and a primary stress in the vertical of $+15$ tons, is the most economic design for the three loadings as shown. This is as economic as the best design for the smaller number of loadings, but usually the economic design for the smaller number of loadings will be the more economical, and, evidently, it will never be the less economical.

This example, perhaps the simplest that could be selected in the design of indeterminate frameworks subject to multiple loadings, will furnish a conception of the difficulty, or rather the practical impossibility, of any general application of such analysis. Here, for single solutions independent of limiting conditions, the solution of quadratic equations was necessary, in one of the cases the solution of a quintic was involved in the determination of economic proportions, and last, but perhaps not least, the examination of numerous cases of defining loadings together with puzzling questions in connection with limiting conditions, had to be gone through with before the most economic design was found and established as such. In any other case the work of this case (which is hardly more than outlined here), and the difficulties would be multiplied many fold. The equations would be of much higher degree and simultaneous, the different cases of defining loadings to be examined would be far more numerous, and the consequences of the limiting conditions would be infinitely more complex and difficult to trace out. In problems of a practical description the quantity and complexity of the work would be such that it is safe to regard it as prohibitive.

Illustration of the Design of an Indeterminate Arch Under Several Loadings.

As a second example consider the two-hinged arch shown in Fig. 1.

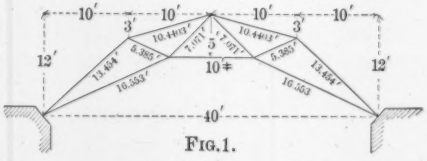


Fig. 1.

The dimensions of the arch are as follows: Span, 40 ft., divided into four parts of 10 ft. each by the upper chord, the rise being 12 ft., and the quarter points being on the parabola through the vertex and the supports. The middle tie is 10 ft. long and 5 ft. from the vertex and all other dimensions follow as shown.

The arch being symmetrical, it is most desirable to denote corresponding bars in a clear manner, which has been done by giving each symmetric pair the same letter, and following it with l or r in subscript to distinguish the bars on the left from those on the right.

The middle bar, which alone has no duplicate, has been denoted by the single letter z (see Fig. 2).

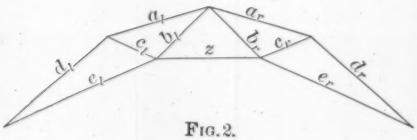


FIG. 2.

The bar z we will treat as the superfluous bar. The stresses S'_{a_r}, S'_{b_r}, etc., due to a tension of unity in z, are as shown in Fig. 3.

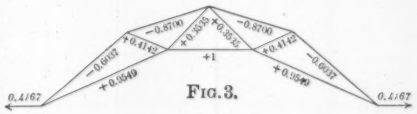


FIG. 3.

We will suppose only the three upper chord joints to be points of loading, neglect dead weight, and consider the various loadings formed by applying the same vertical load P to any one or more of the three points. Denoting the single loadings by L, C or R, according as the load is on the left, center or right joint, we have in single and combined loadings the eight cases, L, C, R, LC, CR, LR, LCR, and "no loading," but of these L and R and LC and CR form symmetrical cases, reducing the loadings requiring separate consideration to six if we confine ourselves to symmetric designs. And only two of these loadings, in addition to the case of "no

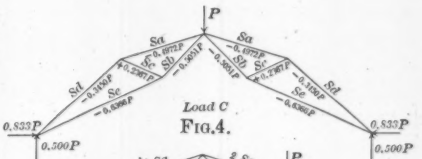


FIG. 4.

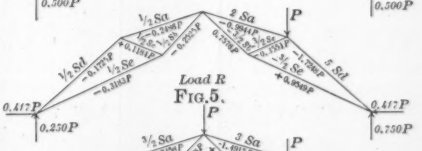


FIG. 5.

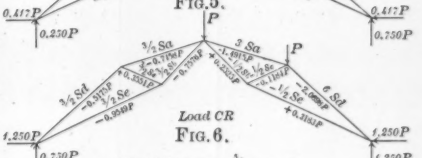


FIG. 6.

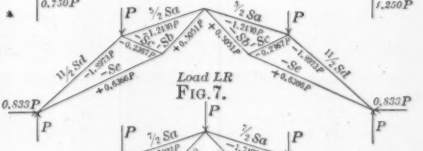


FIG. 7.

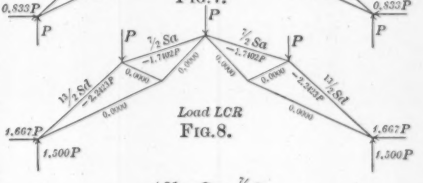


FIG. 8.

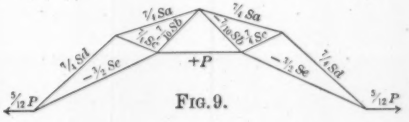


FIG. 9.

loading," require separate calculation, since the others can be obtained through suitably combining these.

The stresses under these various loadings occurring in the statically determined form, that is to say, the three-hinged arch remaining when the middle bar z is omitted, are shown in Figs. 4, 5, 6, 7 and 8.

It will be observed that the stresses of Figs. 6, 7 and 8 are obtained from Figs. 4 and 5 simply by addition.

Denoting the stresses under load C (Fig. 4) by S_a, S_b, S_c, S_d and S_e simply, we find that the stresses under load R (Fig. 5) may be very conveniently expressed in terms of these, and similarly for all the other loadings (Figs. 6, 7 and 8).

Moreover, comparing Fig. 3 with Fig. 4, we also note relations such that if the tension in z were $+P$, then the stresses, in terms of those in Fig. 4, would be as shown in Fig. 9. Therefore, for convenience in the following we will express all stresses in terms of P, which may be omitted from the equations, but which must be taken into consideration when we come to numerical computations.

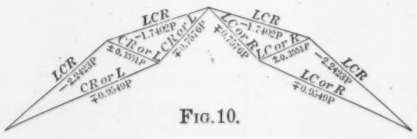


FIG. 10.

Bar	a	b	c	d	e	Total
Volume	$18.166\frac{P}{\sigma}$	$5.357\frac{P}{\sigma}$	$1.912\frac{P}{\sigma}$	$30.174\frac{P}{\sigma}$	$15.806\frac{P}{\sigma}$	$142.830\frac{P}{\sigma}$

Fig. 10 shows the maximum bar stresses of the three-hinged arch together with the loads under which they occur. The bar volumes and total which would correspond to this as a design are also given. We have to find what economy over this can be secured through introducing the middle bar z and making the design indeterminate.

In order to enable us to form a conception of the changes in stress in the indeterminate design, it will be of service to know the changes δz in the middle bar distance under the various loadings in the case of this determinate three-hinged design; for the indeterminate design certainly will have somewhat similar proportions. The shortening of the distance z is $\delta z = \Sigma \frac{S' Sl}{EA}$. Now the areas are the maximum stresses as given in Fig. 10 divided by the working stress; the quotients being taken posi-

tive. Or we may write $\frac{E}{\sigma}\delta z = \Sigma S \frac{S' l}{S_{max}}$. In Fig. 11 are the values of $\frac{S' l}{S_{max}}$ with the signs of the S', and by aid of these the values of $\frac{E}{\sigma}\delta z$ were calculated for all the loadings as shown. The range of $\frac{E}{\sigma}\delta z$ exceeds 60. When the bar z is introduced, that must be so resistent as to reduce the range of $\frac{E}{\sigma}\delta z$ within 20 since z being 10 ft. long, δz may not vary more than $10\frac{\sigma}{E}$ either way from its mean.

Now if we consider the stresses as shown in Figs. 4 to 8, and their modification through introducing a bar z, by aid of the stresses shown in Fig. 3 or Fig. 9, and in the light of this last data as to the character of the variations δz, we see readily that for greatest economy of material in the bars b, c and e the stress in z under L or R must be equal but opposite in sign to that under LC or CR, and similarly under LR equal but of opposite sign to that under C. Therefore the primary stress in z should be equal but of opposite sign to that in z under the full load LCR. This requires that with $\frac{1}{2}P$ on each point the bar z shall be free from stress. The greatest stress in z, and therefore the material used in z in making its greatest strain but $\frac{E}{\sigma}$, would also be least for this condition of design, so that it seems probable that this will furnish a, if not the, most economic, indeterminate design. Only the volumes of the bars a and d would be favored by further increasing the resistance of z under the loading LCR, and it does not seem probable that gains there would offset the consequent losses by the bars b, c, e and z. We will therefore seek the particular solution for which the stress in z would be zero with $\frac{1}{2}P$ on each point, therefore, for which the stress in z under the various loadings gives the following relations: $S_{z_R} = S_{z_L} = -S_{z_{CR}} = -S_{z_{LC}} = S_5$ say, (Fig. 5 corresponding to loading R), $S_{z_{LR}} = -S_{z_C} = S_7$ say, and $S_{z_{LCR}} = -S_{z_0}$ (the primary stress) $= S_8$ say.

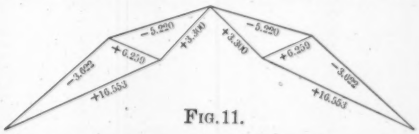


FIG. 11.

Load	C	CR or LR	R or L	LCR	LR	$S_z = P$
$\frac{E}{\sigma}\delta z$	−13.756	+10.328	+24.084	+34.412	+48.168	+52.587

We are now confronted with the question: Under what loadings

will the bars probably have their maximum stresses ? It would seem likely that, as in the three-hinged arch, a and d have their greatest stresses under LCR; b, c and e under R and L (CR and LC); and z, judging by the data of Fig. 11, under LR (and C). We will, therefore, first proceed on this basis. This gives the expressions for the section areas A, and the Figs. 4 to 9 enable us to write the stresses as follows:

$$A_a = \frac{-S_{a_{LCR}}}{\sigma} \qquad S_{a_o} = -\tfrac{7}{4}S_a S_8 \qquad S_{a_{LCR}} = +\tfrac{7}{4}S_a\,(2 + S_8)$$

$$A_b = \frac{S_{b_{rR}}}{\sigma} \qquad S_{b_o} = +\tfrac{7}{10}S_b S_8 \qquad S_{b_{LCR}} = -\tfrac{7}{10}S_b S_8$$

$$A_c = \frac{-S_{c_{rR}}}{\sigma} \qquad S_{c_o} = -\tfrac{7}{4}S_c S_8 \qquad S_{c_{LCR}} = +\tfrac{7}{4}S_c S_8$$

$$A_d = \frac{-S_{d_{LCR}}}{\sigma} \qquad S_{d_o} = -\tfrac{7}{4}S_d S_8 \qquad S_{d_{LCR}} = +\tfrac{1}{4}S_d(26 + 7S_8)$$

$$A_e = \frac{S_{e_{rR}}}{\sigma} \qquad S_{e_o} = +\tfrac{3}{2}S_e S_8 \qquad S_{e_{LCR}} = -\tfrac{3}{2}S_e S_8$$

$$A_z = \frac{-S_{z_{LR}}}{\sigma} = \frac{-S_7}{\sigma}$$

$$S_{a_{LR}} = +\tfrac{1}{4}S_a(10 + 7S_7) \qquad S_{a_{rR}} = +\tfrac{1}{4}S_a(8 + 7S_5)$$

$$S_{b_{LR}} = -\tfrac{1}{10}S_b(10 - 7S_7) \qquad S_{b_{rR}} = -\tfrac{1}{10}S_b(15 + 7S_5)$$

$$S_{c_{LR}} = -\tfrac{1}{4}S_c(4 - 7S_7) \qquad S_{c_{rR}} = -\tfrac{1}{4}S_c(6 - 7S_5)$$

$$S_{d_{LR}} = +\tfrac{1}{4}S_d(22 + 7S_7) \qquad S_{d_{rR}} = +\tfrac{1}{4}S_d(20 + 7S_5)$$

$$S_{e_{LR}} = -\tfrac{1}{2}S_e(2 + 3S_7) \qquad S_{e_{rR}} = -\tfrac{3}{2}S_e(1 + S_5)$$

$$S_{a_C} = +\tfrac{1}{4}S_a(4 - 7S_7) \qquad S_{a_{lCR}} = +\tfrac{1}{4}S_a(6 - 7S_5)$$

$$S_{b_C} = +\tfrac{1}{10}S_b(10 + 7S_7) \qquad S_{b_{lCR}} = +\tfrac{1}{10}S_b(15 + 7S_5)$$

$$S_{c_C} = +\tfrac{1}{4}S_c(4 - 7S_7) \qquad S_{c_{lCR}} = +\tfrac{1}{4}S_c(6 - 7S_5)$$

$$S_{d_C} = +\tfrac{1}{4}S_d(4 - 7S_7) \qquad S_{d_{lCR}} = +\tfrac{1}{4}S_d(6 - 7S_5)$$

$$S_{e_C} = +\tfrac{1}{2}S_e(2 + 3S_7) \qquad S_{e_{lCR}} = +\tfrac{3}{2}S_e(1 + S_5)$$

$$S_{a_{lR}} = +\tfrac{1}{4}S_a(2 + 7S_5) \qquad S_{a_{rCR}} = +\tfrac{1}{4}S_a(12 - 7S_5)$$

$$S_{b_{lR}} = +\tfrac{1}{10}S_b(5 - 7S_5) \qquad S_{b_{rCR}} = -\tfrac{1}{10}S_b(5 - 7S_5)$$

$$S_{c_{lR}} = +\tfrac{1}{4}S_c(2 + 7S_5) \qquad S_{c_{rCR}} = -\tfrac{1}{4}S_c(2 + 7S_5)$$

$$S_{d_{lR}} = +\tfrac{1}{4}S_d(2 + 7S_5) \qquad S_{d_{rCR}} = +\tfrac{1}{4}S_d(24 - 7S_5)$$

$$S_{e_{lR}} = +\tfrac{1}{2}S_e(1 - 3S_5) \qquad S_{e_{rCR}} = -\tfrac{1}{2}S_e(1 - 3S_5)$$

We suppose the coefficient of elasticity to be the same for all bars, therefore, between any two loadings, as $L\,C\,R$ and 0 (no loading), we have equations such as $\Sigma \frac{S'(S_{LCR}-S_o)\,l}{A} = 0$. Writing these for the three pairs of symmetric loadings LCR and 0, R and $C\,R$, and $L\,R$ and C, we have—

$$\frac{-1}{2\sigma}\Sigma\frac{S'(S_{LCR}-S_o)\,l}{A} \equiv \frac{S_a(7+7S_8)\,l_a}{4+2S_8} + \frac{49S_bS_8l_b}{75+35S_5} + \frac{49S_cS_8l_c}{-12+14S_5} + \frac{7S_d(13+7S_8)\,l_d}{52+14S_8} + \frac{3S_eS_8l_e}{1+S_5} + \frac{S_8l_z}{S_7} = 0$$

$$\frac{-1}{2\sigma}\Sigma\frac{S'(S_R-S_{CR})l}{A} \equiv \frac{S_a(-2+7S_8)\,l_a}{4+2S_8} + \frac{7S_b(5+7S_5)\,l_b}{75+35S_5} + \frac{7S_c(-2+7S_5)\,l_c}{-12+14S_5} + \frac{7S_d(-2+7S_5)\,l_d}{52+14S_8} + \frac{S_e(1+3S_5)\,l_e}{1+S_5} + \frac{S_8l_z}{S_7} = 0$$

$$\frac{-1}{2\sigma}\Sigma\frac{S'(S_{LR}-S_C)\,l}{A} \equiv \frac{S_a(3+7S_7)\,l_a}{4+2S_8} + \frac{7S_b(10+7S_7)\,l_b}{75+35S_5} + \frac{7S_c(-4+7S_7)\,l_c}{-12+14S_5} + \frac{7S_d(9+7S_7)\,l_d}{52+14S_8} + \frac{S_e(2+3S_7)\,l_e}{1+S_5} + l_z = 0$$

Now, if to the first of the above equations we add twice the second and then subtract the third, we obtain the simple relation $S_8 + 2S_5 - S_7 = 0$. This we might otherwise have perceived from the following reasoning: The loading $L\,R$ is obtained by adding to the loading $\frac{1}{2}P$, $\frac{1}{2}P$, $\frac{1}{2}P$ the loading $\frac{1}{2}P$,$-\frac{1}{2}P$, $\frac{1}{2}P$. The loading $L\,C\,R$ is obtained by adding to the loading $\frac{1}{2}P$, $\frac{1}{2}P$, $\frac{1}{2}P$, the loading $\frac{1}{2}P$, $\frac{1}{2}P$, $\frac{1}{2}P$. And half the difference of these additive loads (viz., $\frac{1}{2}[(\frac{1}{2}P,-\frac{1}{2}P,\frac{1}{2}P)-(\frac{1}{2}P, \frac{1}{2}P, \frac{1}{2}P)] = (0,-\frac{1}{2}P, 0)$, added to $\frac{1}{2}P$, $\frac{1}{2}P$, $\frac{1}{2}P$ gives the loading $\frac{1}{2}P$, 0, $\frac{1}{2}P$ which would cause the same strain of z as the loading 0, 0, P or loading R. This is a check of the above equations.

Substituting in the second and third of these equations $S_8 + 2S_5$ for S_7 and clearing of fractions in S_8 we get

$$28\,\alpha S^3{}_8 + \begin{bmatrix} (56S_5+160)\,\alpha \\ +14S_a(-2+7S_5)l_a \\ +14S_d(-2+7S_5)l_d \\ +28S_5\,l_z \end{bmatrix} S^2{}_8 + \begin{bmatrix} (320S_5+208)\,\alpha \\ +(28S_5+52)\,S_a(-2+7S_5)l_a \\ +(4S_5+4)\,7S_d(-2+7S_5)l_d \\ +160S_5\,l_z \end{bmatrix} S_8 + \begin{bmatrix} 416S_5\,\alpha \\ +104S_5\,S_a(-2+7S_5)\,l_a \\ +56S_5\,S_d(-2+7S_5)\,l_d \\ +208S_5\,l_z \end{bmatrix} = 0$$

$$\text{where } \alpha \equiv \frac{7S_b\,(5+7S_5)\,l_b}{75+35S_5} + \frac{7S_c\,(-2+7S_5)\,l_c}{-12+14S_5} + \frac{S_e\,(1+3S_5)\,l_e}{1+S_5},$$

and

$$28\,\beta S^3{}_8 + \begin{bmatrix} 160\beta \\ +\ 28\gamma \\ +\ 98S_a\,l_a \\ +\ 98S_d\,l_d \end{bmatrix} S^2{}_8 + \begin{vmatrix} 208\beta \\ +\ 160\gamma \\ +\ 14S_a\,(3+14S_5)\,l_a \\ +\ 14S_d\,(9+14S_5)\,l_d \\ +\ 364S_a\,l_a \\ -+\ 196S_d\,l_d \end{vmatrix} S_8$$

$$+ \begin{bmatrix} 208\gamma \\ +\ 52S_a\,(3+14S_5)\,l_a \\ +\ 28S_d\,(9+14S_5)\,l_d \end{bmatrix} = 0$$

$$\text{where } \beta \equiv \frac{49S_b\,l_b}{75+35S_5} + \frac{49S_c\,l_c}{-12+14S_5} + \frac{3S_e\,l_e}{1+S_5}.$$

$$\gamma \equiv \frac{7S_b\,(10+14S_5)\,l_b}{75+35S_5} + \frac{7S_c\,(-4+14S_5)\,l_c}{-12+14S_5} + \frac{S_e\,(2+6S_5)\,l_e}{1+S_5}$$

$$+ l_z \equiv 2\,\alpha + l_z.$$

The coefficients of S_8 in each of the above cubics in S_8 contain the variable S_5 and otherwise only known quantities. Between the two cubics we could eliminate S_5, obtaining a relation between the coefficients which, cleared of fractions, would yield an equation of about the twentieth degree in S_5. This we could then solve directly. Or, by assuming a series of values of S_5 and determining the corresponding values of S_8 from the cubics, we can plot the curves in part for each of the above equations, and their intersections will furnish the common roots. Adopting this latter method, we find—

if $S_5 = 0$	$S^3{}_8 + 4.798S^2{}_8 + 4.764\,S_8 + 0 = 0$	has root 0
	$S^3{}_8 + 6.885S^2{}_8 + 12.385\,S_8 + 3.975 = 0$	" " -0.406
" $S_5 = -0.1$	$S^3{}_8 + 3.735S^2{}_8 + 1.912\,S_8 + 3.679 = 0$	has root -3.49
	$S^3{}_8 + 6.62\,S^2{}_8 + 10.83\,S_8 + 1.870 = 0$	" " -0.195
" $S_5 = -0.03$	$S^3{}_8 + 4.588S^2{}_8 + 4.139\,S_8 + 0.479 = 0$	has root $-0\,135$
	$S^3{}_8 + 6.820S^2{}_8 + 12.000\,S_8 + 3.441 = 0$	" " -0.354
" $S_5 = -0.05$	$S^3{}_8 + 4.434S^2{}_8 + 3.702\,S_8 + 0.781 = 0$	has root -0.336
	$S^3{}_8 + 6.778S^2{}_8 + 11.743\,S_8 + 3.094 = 0$	" " -0.320

Plotting these, we find an intersection of the curve of the second equation with that of the first at $S_5 = -0.049 \quad S_8 = -0.322$, which is

approximately the solution sought. From these values of S_5 and S_8 we get $S_7 = S_8 + 2S_5 = -0.420$.

But this gives us $\begin{matrix} S_{c_{LR}} = -\frac{1}{4} S_c (4 + 7 \times 0.420) = -\frac{1}{4} S_c \times 6.940 \\ S_{c_{rR}} = -\frac{1}{4} S_c (6 + 7 \times 0.049) = -\frac{1}{4} S_c \times 6.343 \end{matrix}$

or $S^2_{c_{LR}} > S^2_{c_{rR}}$ which is contrary to our supposition that c has its maximum stress and is designed under the loading R. All other bars have their maximum stresses as supposed. We must therefore substitute $A_c = \frac{-S_{c_{LR}}}{\sigma}$ for its previous value, rewrite our equations and solve again. This affects only one denominator of our equations (page 391), which thus changed become (the last two only being written),

$$\frac{-1}{2\sigma} \Sigma \frac{S'(S_R - S_{cR}) l}{A} \equiv \frac{S_a(-2+7S_5) l_a}{4+2S_8} + \frac{7 S_b (5+7 S_5) l_b}{75+35 S_5}$$

$$+ \frac{7 S_c (-2+7S_5) l_c}{-8+14S_7} + \frac{7 S_d (-2+7 S_5) l_d}{52+14S_8} + \frac{S_e (1+3 S_5) l_e}{1+S_5} + \frac{S_5 l_z}{S_7} = 0$$

$$\frac{-1}{2\sigma} \Sigma \frac{S'(S_{LR} - S_C) l}{A} \equiv \frac{S_a (3+7 S_7) l_a}{4+2 S_8} + \frac{7 S_b (10+7 S_7) l_b}{75+35 S_5} + \frac{7 S_c l_c}{2}$$

$$+ \frac{7 S_d (9+7 S_7) l_d}{52+14 S_5} + \frac{S_e (2+3 S_7) l_e}{1+S_5} + l_z = 0$$

Making the substitution $S_8 + 2 S_5$ for S_7 and clearing of fractions in S_8 as before, we obtain two equations, the first a biquadratic, the second a cubic in S_8. We have

$$\begin{bmatrix} 392\,\alpha' \\ -686\, S_c\, l_c \end{bmatrix} S^4_8 + \begin{bmatrix} (1568 S_5 + 2016)\,\alpha' \\ -49 (28 S_5 + 80) S_c l_c \\ +196 (-2+7 S_5) S_a l_a \\ +196 (-2+7 S_5) S_d l_d \\ +392 S_5 l_z \end{bmatrix} S^3_8$$

$$+ \begin{bmatrix} (1568 S^2_5 + 8512 S_5 + 1632)\,\alpha' \\ -49 (160 S_5 + 104) S_c l_c \\ +(784 S_5 + 616)(-2+7 S_5) S_a l_a \\ +7 (112 S_5 + 40)(-2+7 S_5) S_d l_d \\ +(784 S_5 + 2016) S_5 l_z \end{bmatrix} S^2_8$$

$$+\begin{bmatrix}(8960\,S^2_5+9088\,S_5-1664)\,\alpha'\\ -10192\,S_5\,S_c\,l_c\\ +(784\,S^2_5+2688\,S_5-416)(-2+7\,S_5)S_a l_a\\ +7(112\,S^2_5+192\,S_5-32)(-2+7S_5)S_d\,l_d\\ +(4480\,S_5+1632)\,S_5\,l_z\end{bmatrix}S_8$$

$$+\begin{bmatrix}(11648\,S_5-3328)\,S_5\,\alpha'\\ +(2912\,S_5-832)\,S_5\,(-2+7\,S_5)\,S_a\,l_a\\ +7\,(224\,S_5-64)\,S_5\,(-2+7\,S_5)\,S_d\,l_d\\ +(5824\,S_5-1664)\,S_5\,l_z\end{bmatrix}=0$$

where $\alpha' \equiv \dfrac{7\,S_b\,(5+7\,S_5)\,l_b}{75+35\,S_5}+\dfrac{7\,S_c\,l_c}{2}+\dfrac{S_e\,(1+3\,S_5)\,l_e}{1+S_5}$

and

$$28\beta'\,S^3_8+\begin{bmatrix}160\,\beta'\\ +28\,\gamma'\\ +98\,S_a\,l_a\\ +98\,S_d\,l_d\end{bmatrix}S^2_8+\begin{bmatrix}208\,\beta'\\ +160\,\gamma'\\ +14(3+14\,S_5)\,S_a\,l_a\\ +14(9+14\,S_5)\,S_d\,l_d\\ +364\,S_a\,l_a\\ +196\,S_d\,l_d\end{bmatrix}S_8$$

$$+\begin{bmatrix}208\,\gamma'\\ +52\,(3+14\,S_5)\,S_a\,l_a\\ +28\,(9+14\,S_5)\,S_d\,l_d\end{bmatrix}=0$$

where $\beta' \equiv \dfrac{49\,S_b\,l_b}{75+35\,S_5}+\dfrac{3\,S_e\,l_e}{1+S_5}$ $\gamma' \equiv 2\,\alpha'+l_z$.

Solving these equations just as in the preceding case we have the following:

If $S_5=-0.05$ $S^4_8+4.394\,S^3_8+1.856\,S^2_8-1.972\,S_8-0.0309=0$ has root -0.015
$S^3_8+6.891\,S^2_8+12.151\,S_8+3.278=0$ " " -0.328

" $S_5=-0.10$ $S^4_8+3.655\,S^3_8+0.8741\,S^2_8-1.2567\,S_8-0.3216=0$ has root -0.2558
$S^3_8+6.761\,S^2_8+11.420\,S_8+2.325=0$ " " -0.2352

" $S_5=-0.098$ $S^4_8+3.674\,S^3_8+0.9106\,S^2_8-1.2996\,S-0.3022=0$ has root -0.2275
$S^3_8+6.767\,S^2_8+11.450\,S_8+2.363=0$ " " -0.2393

Graphically interpolating, we find approximately $S_5=-0.0990$, $S_8=-0.2373$, which gives $S_7=S_8+2S_5=-0.4353$.

Applying the formulas for the bar stresses written out on page 390 we obtain the series of stresses under the various loadings shown here in figures 4*a* to 8*a* and *a* (compare figures 4 to 8, page 387). But we observe that, contrary to our suppositions, the greatest stresses in the bars *a* and *d* do not occur under *LCR* but under *CR* (and *LC*). Under this latter loading the upper chord stresses are $1\frac{1}{2}$ to $2\frac{1}{2}\%$ greater than under the full loading. The design is not, therefore, admissible, but since it is a close approximation it will be of interest to note the volume of material it takes and compare with that for the three-hinged arch. The section areas are shown in Fig. 12, below which are the bar volumes. In spite of the overstrain of the bars *a* and *d* under the loadings *CR* and *LC*, the economy is but some $4\frac{1}{2}\%$ over the three-hinged arch (see Fig. 10, page) 388.

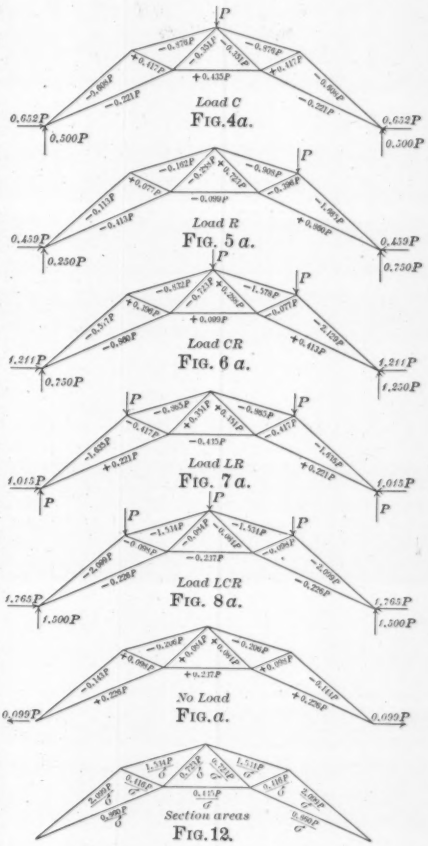


FIG. 4*a*.

FIG. 5*a*.

FIG. 6*a*.

FIG. 7*a*.

FIG. 8*a*.

FIG. *a*.

FIG. 12.

Bar	*a*	*b*	*c*
Volume	$16.01\frac{P}{\sigma}$	$5.11\frac{P}{\sigma}$	$2.25\frac{P}{\sigma}$

d	*e*	*z*	Total
$28.24\frac{P}{\sigma}$	$14.24\frac{P}{\sigma}$	$4.35\frac{P}{\sigma}$	$136.05\frac{P}{\sigma}$

We have now to solve on the supposition that all the bars except *c* and *z* have their greatest stresses under the loadings *CR* and *LC*, so that now

$$A_a = \frac{-S_{a_{r_{CR}}}}{\sigma} = \frac{-\frac{1}{4} S_a (12 - 7 S_5)}{\sigma}$$

$$A_d = \frac{-S_{d_{r_{CR}}}}{\sigma} = \frac{-\frac{1}{4} S_d (24 - 7 S_5)}{\sigma}.$$

We have, therefore,

$$\frac{-1}{2\sigma} \Sigma \frac{S' (S_R - S_{CR}) l}{A} \equiv \frac{7 S_a (-2 + 7 S_5) l_a}{24 - 14 S_5} + \frac{7 S_b (5 + 7 S_5) l_b}{75 + 35 S_5}$$

$$+ \frac{7 S_c (2 - 7 S_5) l_c}{8 - 14 S_7} + \frac{7 S_d (-2 + 7 S_5) l_d}{48 - 14 S_5} + \frac{S_e (1 + 3 S_5) l_e}{1 + S_5} + \frac{S_5 l_z}{S_7} = 0.$$

$$\frac{-1}{2\sigma} \Sigma \frac{S' (S_{LR} - S_C) l}{A} \equiv \frac{7 S_a (3 + 7 S_7) l_a}{24 - 14 S_5} + \frac{7 S_b (10 + 7 S_7) l_b}{75 + 35 S_5}$$

$$+ \frac{7 S_c l_c}{2} + \frac{7 S_d (9 + 7 S_7) l_d}{48 - 14 S_5} + \frac{S_e (2 + 3 S_7) l_e}{1 + S_5} + l_z = 0.$$

These contain only the variables S_5 and S_7. Writing out in terms of S_7 we obtain the equations

$$14\alpha'' S^2_7 + \begin{bmatrix} -8\alpha'' \\ -7(2 - 7 S_5) S_c l_c \\ +14 S_5 l_z \end{bmatrix} S_7 - 8 S_5 l_z = 0 \text{ and } \beta'' S_7 + \gamma'' = 0.$$

where

$$\alpha'' \equiv \frac{7 S_a (-2 + 7 S_5) l_a}{24 - 14 S_5} + \frac{7 S_b (5 + 7 S_5) l_b}{75 + 35 S_5} + \frac{7 S_d (-2 + 7 S_5) l_d}{48 - 14 S_5}$$

$$+ \frac{S_e (1 + 3 S_5) l_e}{1 + S_5}.$$

$$\beta'' \equiv \frac{49 S_a l_a}{24 - 14 S_5} + \frac{49 S_b l_b}{75 + 35 S_5} + \frac{49 S_d l_d}{48 - 14 S_5} + \frac{3 S_e l_e}{1 + S_5},$$

$$\gamma'' \equiv \frac{21 S_a l_a}{24 - 14 S_5} + \frac{70 S_b l_b}{75 + 35 S_5} + \frac{7 S_c l_c}{2} + \frac{63 S_d l_d}{48 - 14 S_5} + \frac{2 S_e l_e}{1 + S_5} + l_z.$$

Here our equations are much simpler than before, one being linear and the other quadratic in S_7. We could readily eliminate S_7, obtaining an equation of the eleventh degree in S_5, but it will be simpler to proceed as before by assuming values of S_5, computing S_7 and graphically obtaining the common roots.

If $S_5 = -0.100$ $\quad S^2_7 + 0.098495\ S_7 - 0.14068 = 0$ has root -0.4277
$\quad S_7 + 0.4345 = 0$ has root -0.4345.

If $S_5 = -0.101$ $\quad S^2_7 + 0.1087\ S_7 - 0.14349 = 0$ has root -0.4370
$\quad S_7 + 0.4348 = 0$ has root -0.4348.

Graphically interpolating we find

$S_5 = -0.1008.$ $\quad S_7 = -0.4346.$ $\quad S_8 = S_7 - 2 S_5 = -0.2330.$

We obtain through the formulas for bar stresses on page 390 the series of stresses under the various loadings shown here in Figs. 4*b* to 8*b* and *b*.

This time the greatest stresses all occur under the loads as assumed, and the solution here obtained is therefore that sought.

The section areas are as given in Fig. 13, together with the loadings under which they are designed. And below are given the volumes of the bars and the total volume (compare Fig. 10, page 388). The economy thus obtained over the three-hinged arch proves to be but $3\frac{1}{2}$ per cent.

We may test the work by computing the shortening of the line z from its length when the bars a, b, c, d and e all have zero strain. We have $\frac{E}{\sigma}\delta z = \Sigma \frac{S'l}{S_{max}} S$ (if we write S_{max} always $+$). The values of $\frac{S'l}{S_{max}}$ being as shown in Fig. 14, we obtain, by aid of the stresses given in Figs. 4*b* to 8*b* and *b*, the values of $\frac{E}{\sigma}\delta z$ shown above.

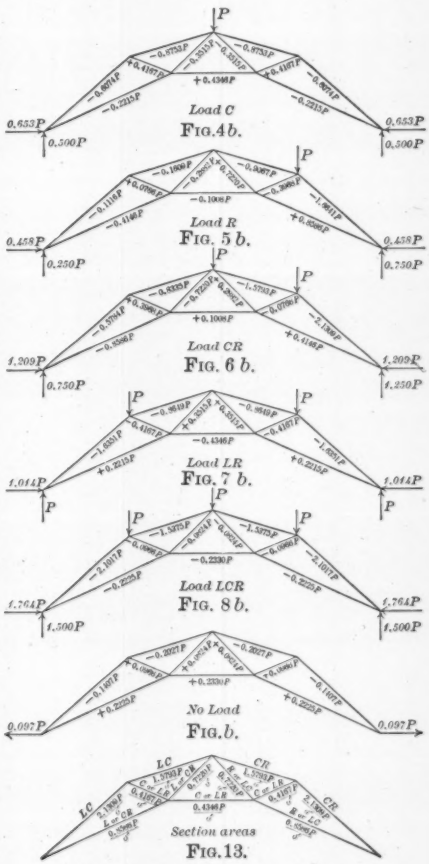


FIG. 4*b*. FIG. 5*b*. FIG. 6*b*. FIG. 7*b*. FIG. 8*b*. FIG. *b*. FIG. 13.

Bar	*a*	*b*	*c*	*d*
Volume	$16.49\frac{P}{\sigma}$	$5.11\frac{P}{\sigma}$	$2.24\frac{P}{\sigma}$	$28.67\frac{P}{\sigma}$

Bar	*e*	*z*	Total
Volume	$14.21\frac{P}{\sigma}$	$4.34\frac{P}{\sigma}$	$137.78\frac{P}{\sigma}$

Subtracting from the others the value for $\frac{1}{2}(LCR)$ when the bar z has no strain, we obtain $\frac{E}{\sigma}$ times the actual strain of z. This should be ± 10 (more accurately $\pm l_z$ which is somewhat less than 10 ft.), for loads LR and C, respectively, $\pm \frac{1008}{4346} \times 10 = \pm 2.319$ for R and CR, respectively, and $\pm \frac{2330}{4346} \times 10 = \pm 5.361$ for LCR and "no load," respectively. Even these, which are a trifle too large owing to the value of l_z used, check within $\frac{2}{10}\%$, or within the probable error from keeping but four significant figures in the calculations.

The normal or unstrained length of the bar z is its length under the loading which makes its stress zero, that is $\frac{1}{2}P$ on each panel point. This length is $(10 - 18.556 \frac{\sigma}{E}$ ft.). Taking $\sigma = 10\,000$ lbs. per square inch, and $E = 30\,000\,000$ lbs. per square inch, this correction is 0.00618 ft., the corrected length of z being 9.99382 ft. It is seen that an error of a ten-thousandth of a foot would sensibly change the strain, and the same is true of the other bars. To ensure the distribution of the stresses as here calculated, within, say, 1%, the bar lengths should all be calculated to five decimal places and single errors in the finished structure should not much exceed a unit in the last decimal place. That is, the bar lengths should be true to about one ten-thousandth of an inch; certainly closer than to one-thousandth of an inch. And we must further suppose no comparable yielding of joints or abutments.

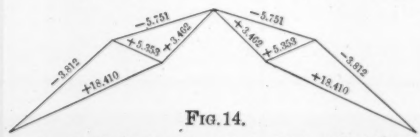


FIG. 14.

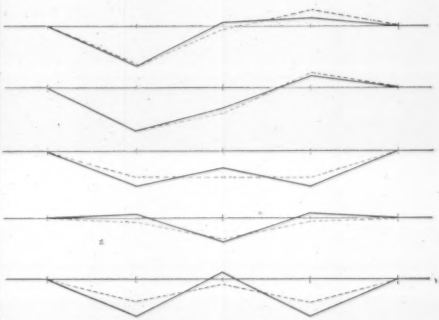
Load	C	*No Load*	CR	R	LCR	LR	$\frac{1}{2}(LCR)$
$\frac{E}{\sigma}\delta z$	+8.572	+13.198	+16.242	+20.871	+23.912	+28.540	+18.556
	−9.984	−5.358	−2.314	+2.315	+5.356	+9.984	0.000

The relative stiffness of the indeterminate design just obtained as compared with that of the determinate (three-hinged) design is a matter of much interest, since it is frequently claimed that the two-hinged arch is stiffer than the three-hinged arch. If by S' we indicate the stresses in the three-hinged arch due to a vertical load of unity on any joint and by S the actual stresses in the bars under any given loading, in either design, then $\Sigma \frac{S'Sl}{EA}$ will give the sinking of the joint in question under the given loading. The values of S' for load

unity on the middle joint or on the right upper chord joint we readily obtain from Figs. 4 and 5, the values of S we obtain from Figs. 4 to 8 and Figs. 4b to 8b and b, and the values of A from Fig. 10 and Fig. 13. We note that for the indeterminate design "no load" also gives deflections, $-3.72\ \frac{\sigma}{E}$ at the middle joint and $+6.19\ \frac{\sigma}{E}$ at the side upper chord joints. The deflections, corrected as above for the indeterminate design and without the factor $\frac{\sigma}{E}$, are as follows:

Load	Joint	Deflection: Three-hinged arch.	Deflection: Two-hinged arch.
L	L	+49.96	+44.64
	C	−2.63	+2.52
	R	−8.84	−18.40
LC	L	+47.33	+47.18
	C	+22.29	+26.12
	R	−11.47	−15.86
LCR	L	+38.50	+28.79
	C	+19.66	+28.64
	R	+38.50	+28.79
C	L	−2.63	+2.54
	C	+24.92	+23.59
	R	−2.63	+2.54
LR	L	+41.13	+26.24
	C	−5.25	+5.05
	R	+41.13	+26.24

Plotted Deflections:
Full line for Three-hinged arch.
Dotted " " Two- " "



We note that the deflections differ in character in the two cases. The greatest deflections do not occur under the same loadings. The greatest side joint deflection of the three-hinged arch is some 6% greater than that of the two-hinged arch, but the greatest center joint deflection of the three-hinged arch is some 15% less than that of the two-hinged arch. The center deflections are little more than half the quarter deflections in both cases. Practically one arch is about as stiff as the other. Certainly the existence of the middle bar in the two-hinged arch does not appear to have given it a pronounced advantage.

Is the indeterminate solution which we have found one of greatest economy of material? It at least is a solution at a point of discontinuity in the variation of the material used. That is, under our condition of equal maximum stress, if the compression in z under R were slightly less, the tension in z under $C\ R$ would be slightly greater, A_b and A_e would be determined under the loading R, and A_c and A_z would be determined under the loading C. But if the compression

in z under R were slightly greater, the tension in z under $C\,R$ would be slightly less, A_b and A_e would be determined under the loading $C\,R$ and A_c and A_z would be determined under the loading $L\,R$. For, consider the three equations between R and $C\,R$, $L\,R$ and C, and R and $L\,R$ in the former case. Put $S_{z_R} = S_5 + \Delta_5$, $S_{z_{CR}} = -S_5 + \Delta_6$, $S_{z_{LR}} = S_7 + \Delta_7$ and $S_{z_C} = -S_7 + \Delta_4$, where Δ_5 is $+$. Then

$$
\begin{aligned}
&-\frac{1}{\sigma}\Sigma\frac{S'(S_R - S_{CR})\,l}{A}\\
&\equiv \frac{7\,S_a(-4+14\,S_5+7\Delta_5-7\Delta_6)\,l_a}{24-14\,S_5+14\Delta_6}\\
&+\frac{7\,S_b(10+14\,S_5+7\Delta_5-7\Delta_6)\,l_b}{75+35\,S_5+35\Delta_5}\\
&+\frac{7\,S_c(4-14\,S_5-7\Delta_5+7\Delta_6)\,l_c}{8-14\,S_7+14\Delta_4}\\
&+\frac{7\,S_d(-4+14\,S_5+7\Delta_5-7\Delta_6)\,l_d}{48-14\,S_5+14\Delta_6}\\
&+\frac{S_e(2+6\,S_5+3\Delta_5-3\Delta_6)\,l_e}{1+S_5+\Delta_5}\\
&+\frac{(2S_5+\Delta_5-\Delta_6)\,l_z}{S_7-\Delta_4}\\
&=0
\end{aligned}
$$

$$
\begin{aligned}
&-\frac{1}{\sigma}\Sigma\frac{S'(S_{LR} - S_C)\,l}{A}\\
&\equiv \frac{7\,S_a(6+14\,S_7+7\Delta_7-7\Delta_4)\,l_a}{24-14\,S_5+14\Delta_6}\\
&+\frac{7\,S_b(20+14\,S_7+7\Delta_7-7\Delta_4)\,l_b}{75+35\,S_5+35\Delta_5}\\
&+\frac{7\,S_c(8-14\,S_7-7\Delta_7+7\Delta_4)\,l_c}{8-14\,S_7+14\Delta_4}\\
&+\frac{7\,S_d(18+14\,S_7+7\Delta_7-7\Delta_4)\,l_d}{48-14\,S_5+14\Delta_6}\\
&+\frac{S_e(4+6\,S_7+3\Delta_7-3\Delta_4)\,l_e}{1+S_5+\Delta_5}\\
&+\frac{(2S_7+\Delta_7-\Delta_4)\,l_z}{S_7-\Delta_4}\\
&=0
\end{aligned}
$$

$$
\begin{aligned}
&-\frac{1}{\sigma}\Sigma\frac{S'(S_{LR} - S_R)\,l}{A}\\
&\equiv \frac{7\,S_a(5+7\,S_7-7\,S_5+7\Delta_7-7\Delta_5)\,l_a}{24-14\,S_5+14\Delta_6}\\
&+\frac{7\,S_b(5+7\,S_7-7\,S_5+7\Delta_7-7\Delta_5)\,l_b}{75+35\,S_5+35\Delta_5}\\
&+\frac{7\,S_c(2-7\,S_7+7\,S_5-7\Delta_7+7\Delta_5)\,l_c}{8-14\,S_7+14\Delta_4}\\
&+\frac{7\,S_d(11+7\,S_7-7\,S_5+7\Delta_7-7\Delta_5)\,l_d}{48-14\,S_5+14\Delta_6}\\
&+\frac{S_e(1+3\,S_7-3\,S_5+3\Delta_7-3\Delta_5)\,l_e}{1+S_5+\Delta_5}\\
&+\frac{(S_7-S_5+\Delta_7-\Delta_5)\,l_z}{S_7-\Delta_4}\\
&=0
\end{aligned}
$$

Remembering that the Δ are all very small, and noting the two equations on page 396 and also the equation formed by their difference, the equations just written may be reduced to:

$$-\frac{1}{\sigma}\Sigma\frac{S'(S_R-S_{CR})\,l}{A}$$

$$=\frac{7S_a(7\Delta_5-7\Delta_6+\frac{4-14S_5}{24-14S_5}\times 14\,\Delta_6)\,l_a}{24-14S_5}$$

$$+\frac{7S_b(7\Delta_5-7\Delta_6-\frac{10+14S_5}{75+35S_5}\times 35\Delta_5)\,l_b}{75+35S_5}$$

$$+\frac{7S_c(-7\Delta_5+7\Delta_6-\frac{4-14S_5}{8-14S_7}\times 14\Delta_4)\,l_c}{8-14S_7}$$

$$+\frac{7S_d(7\Delta_5-7\Delta_6+\frac{4-14S_5}{48-14S_5}\times 14\Delta_6)\,l_d}{48-14S_5}$$

$$+\frac{S_e(3\Delta_5-3\Delta_6-\frac{2+6S_5}{1+S_5}\times\Delta_5)\,l_e}{1+S_5}$$

$$+\frac{(\Delta_5-\Delta_6+\frac{2S_5}{S_7}\times\Delta_4)\,l_z}{S_7}$$

$$=0$$

$$-\frac{1}{\sigma}\Sigma\frac{S'(S_{LR}-S_C)\,l}{A}$$

$$=\frac{7S_a(7\Delta_7-7\Delta_4-\frac{6+14S_7}{24-14S_5}\times 14\Delta_6)\,l_a}{24-14S_5}$$

$$+\frac{7S_b(7\Delta_7-7\Delta_4-\frac{20+14S_7}{75+35S_5}\times 35\Delta_5)\,l_b}{75+35S_5}$$

$$+\frac{7S_c(-7\Delta_7+7\Delta_4-14\Delta_4)\,l_c}{8-14S_7}$$

$$+\frac{7S_d(7\Delta_7-7\Delta_4-\frac{18+14S_7}{48-14S_5}\times 14\Delta_6)\,l_d}{48-14S_5}$$

$$+\frac{S_e(3\Delta_7-3\Delta_4-\frac{4+6S_7}{1+S_5}\times\Delta_5)\,l_e}{1+S_5}$$

$$+\frac{(\Delta_7-\Delta_4+2\Delta_4)\,l_z}{S_7}$$

$$=0$$

$$-\frac{1}{\sigma}\Sigma\frac{S'\,(S_{LR}-S_R)\,l}{A}$$

$$=\frac{7S_a(7\Delta_7-7\Delta_5-\frac{5+7S_7-7S_5}{24-14S_5}\times 14\Delta_6)\,l_a}{24-14S_5}$$

$$+\frac{7S_b\,(7\Delta_7-7\Delta_5-\frac{5+7S_7-7S_5}{75+35S_5}\times 35\Delta_5)\,l_b}{75+35S_5}$$

$$+\frac{7S_c(-7\Delta_7+7\Delta_5-\frac{2-7S_7+7S_5}{8-14S_7}\times 14\Delta_4)\,l_c}{8-14S_7}$$

$$+\frac{7S_d(7\Delta_7-7\Delta_5-\frac{11+7S_7-7S_5}{48-14S_5}\times 14\Delta_6)\,l_d}{48-14S_5}$$

$$+\frac{S_e(3\Delta_7-3\Delta_5-\frac{1+3S_7-3S_5}{1+S_5}\times\Delta_5)\,l_e}{1+S_5}$$

$$+\frac{(\Delta_7-\Delta_5+\frac{S_7-S_5}{S_7}\times\Delta_4)\,l_z}{S_7}$$

$$=0.$$

These are three linear equations between the four quantities Δ_4, Δ_5, Δ_6 and Δ_7.

Putting in the numerical values of S_a, S_b, S_c, S_d, S_e, l_a, l_b, l_c, l_d, l_e, l_z, and of S_5 and S_7, these equations may be reduced to the numerical equations:

$$\left.\begin{aligned} &\Delta_4 + 4.26\Delta_5 - 5.29\Delta_6 = 0 \\ &\Delta_4 + 0.82\Delta_5 + 0.09\Delta_6 - 3.21\Delta_7 = 0 \\ &\Delta_4 - 3.92\Delta_5 - 0.18\Delta_6 + 3.90\Delta_7 = 0 \end{aligned}\right\} \text{ which give } \left\{\begin{aligned} \Delta_5 &= +1.4\Delta_7 \\ \Delta_6 &= +1.5\Delta_7 \\ \Delta_4 &= +1.9\Delta_7 \end{aligned}\right.$$

Thus the Δ are all + as stated. But this change then increases all of the designing stresses $S_{a_{r_{CR}}}$, $S_{b_{r_R}}$, S_{c_C}, $S_{d_{r_{CR}}}$, $S_{e_{r_R}}$, and S_{z_C}, and therefore increases the material used.

Now, consider the similar equations between the loadings R and CR, LR and C, and R and LR in the second case, when Δ_5 is —, and a, b, d and e are designed under CR, and c and z under LR. We may at once write them down in the second form, the change being simply replacing Δ_5 by $-\Delta_6$ and Δ_4 by $-\Delta_7$ where, in the brackets, these have a fraction for a coefficient. We have:

$$-\frac{1}{\sigma}\Sigma\,\frac{S'(S_R - S_{CR})l}{A}$$

$$= \frac{7S_a(7\Delta_5 - 7\Delta_6 + \frac{4-14S_5}{24-14S_5}\times 14\Delta_6)\,l_a}{24-14S_5}$$

$$+ \frac{7S_b(7\Delta_5 - 7\Delta_6 + \frac{10+14S_5}{75+35S_5}\times 35\Delta_6)l_b}{75+35S_5}$$

$$+ \frac{7S_c(-7\Delta_5 + 7\Delta_6 + \frac{4-14S_5}{8-14S_7}\times 14\Delta_7)l_c}{8-14S_7}$$

$$+ \frac{7S_d(7\Delta_5 - 7\Delta_6 + \frac{4-14S_5}{48-14S_5}\times 14\Delta_6)l_d}{48-14S_5}$$

$$+ \frac{S_e(3\Delta_5 - 3\Delta_6 + \frac{2+6S_5}{1+S_5} + \Delta_6)\,l_e}{1+S_5}$$

$$+ \frac{(\Delta_5 - \Delta_6 - \frac{2S_5}{S_7} + \Delta_7)\,l_z}{S_7}$$

$$= 0$$

$$-\frac{1}{\sigma}\Sigma\,\frac{S'(S_{LR} - S_C)\,l}{A}$$

$$= \frac{7S_a(7\Delta_7 - 7\Delta_4 - \frac{6+14S_7}{24-14S_5}\times 14\Delta_6)\,l_a}{24-14S_5}$$

$$+ \frac{7S_b(7\Delta_7 - 7\Delta_4 + \frac{20+14S_7}{75+35S_5}\times 35\Delta_6)\,l_b}{75+35S_5}$$

$$+ \frac{7S_c(-7\Delta_7 + 7\Delta_4 + 14\Delta_7)\,l_c}{8-14S_7}$$

$$+ \frac{7S_d(7\Delta_7 - 7\Delta_4 - \frac{18+14S_7}{48-14S_5}\times 14\Delta_6)\,l_d}{48-14S_5}$$

$$+ \frac{S_e(3\Delta_7 - 3\Delta_4 + \frac{4+6S_7}{1+S_5}\times\Delta_6)\,l_e}{1+S_5}$$

$$+ \frac{(\Delta_7 - \Delta_4 - 2\Delta_7)\,l_z}{S_7}$$

$$= 0$$

$$-\frac{1}{\sigma}\Sigma\frac{S'(S_{LR}-S_R)\,l}{A}$$

$$=\frac{7S_a\,(7\Delta_7-7\Delta_5-\dfrac{5+7S_7-7S_5}{24-14S_5}\times 14\Delta_6)\,l_a}{24-14S_5}$$

$$+\frac{7S_b\,(7\Delta_7-7\Delta_5+\dfrac{5+7S_7-7S_5}{75+35S_5}\times 35\Delta_6)\,l_b}{75+35S_5}$$

$$+\frac{7S_c\,(-7\Delta_7+7\Delta_5+\dfrac{2-7S_7+7S_5}{8-14S_7}\times 14\Delta_7)\,l_c}{8-14S_7}$$

$$+\frac{7S_d\,(7\Delta_7-7\Delta_5-\dfrac{11+7S_7-7S_5}{48-14S_5}\times 14\Delta_6)\,l_d}{48-14S_5}$$

$$+\frac{S_e(3\Delta_7-3\Delta_5+\dfrac{1+3S_7-3S_5}{1+S_5}\times\Delta_6)\,l_e}{1+S_5}$$

$$+\frac{(\Delta_7-\Delta_5-\dfrac{S_7-S_5}{S_7}\times\Delta_7)\,l_z}{S_7}$$

$$=0$$

These give three linear equations between the four quantities Δ_4, Δ_5, Δ_6 and Δ_7, which may be reduced to the numerical equations:

$$\left.\begin{aligned}\Delta_5-0.687\Delta_6-0.177\Delta_7&=0\\ \Delta_4-0.230\Delta_6-0.313\Delta_7&=0\\ \Delta_5+0.041\Delta_6-0.744\Delta_7&=0\end{aligned}\right\}\text{ which give....}$$

$$\text{.....}\left\{\begin{aligned}\Delta_6&=+0.78\Delta_7 \quad \Delta_7=\frac{+1}{0.78}\quad \Delta_6=+1.28\Delta_6\\ \Delta_5&=+0.71\Delta_7\\ \Delta_4&=+0.49\Delta_7\end{aligned}\right.$$

Thus the Δ are all of the same sign, or negative, and a, b, d and e have their greatest stresses under CR, and c and z theirs under LR, as stated. The changes in volume due to a change Δ_6 (increased compression in z under CR) are:

Volume of b increases $S'_b l_b \Delta_6 \frac{1}{\sigma} = 2.50 \Delta_6 \frac{1}{\sigma}$

" " e " $S'_e l_e \Delta_6 \frac{1}{\sigma} = 15.80 \Delta_6 \frac{1}{\sigma}$

" " c " $S'_c l_c \Delta_7 \frac{1}{\sigma} = 2.85 \Delta_6 \frac{1}{\sigma}$

$21.15 \Delta_6 \frac{1}{\sigma}$

2

" " $2(b + c + e)$ increases $= 42.30 \Delta_6 \frac{1}{\sigma}$

" " z " $l_z \Delta_7 \frac{1}{\sigma} = 12.80 \Delta_6 \frac{1}{\sigma}$

" " $2(b + c + e) + z$ increases $55.10 \Delta_6 \frac{1}{\sigma}$

Volume of a diminishes $S'_a l_a \Delta_6 \frac{1}{\sigma} = 9.08 \Delta_6 \frac{1}{\sigma}$

" " d " $S'_d l_d \Delta_6 \frac{1}{\sigma} = 8.12 \Delta_6 \frac{1}{\sigma}$

$17.20 \Delta_6 \frac{1}{\sigma}$

2

" " $2(a + d)$ diminishes $= 34.40 \Delta_6 \frac{1}{\sigma}$

Thus this variation also increases the material used, the saving of material on the bars a and d being considerably more than offset by the increased material in the bars b, c, e and z. Thus the indeterminate solution we have found is at a point of discontinuity which is a point of greatest economy. Now it is readily shown that a larger primary stress in z is always less economic, and simple investigation shows that the smaller the primary tension in z and therefore the greater its greatest compression, the less saved on the bars a and d compared with that expended on the bars b, c, e and z. This solution is therefore the most economic symmetric solution; and as investigation shows that unsymmetric solutions are less economic than the symmetric, we may conclude that the design obtained is the most economic solution of the given two-hinged arch for the given loadings, on the basis of a uniform greatest mean stress in designing.

Now, the economic superiority of this most economic indeterminate design over the simple determinate design (the three-hinged arch), is but some 3½% (see page 397). Such a small saving, if all other things were equal, would hardly be worth the labor of seeking; but we recognize that, aside from the difficulty of determining the design, other things are not at all equal, for, to actually realize the stress distribution calculated, so accurately as to really preserve the 3½% advantage, extremely accurate and therefore expensive work would be necessary, and also very rigid and therefore expensive abutments. Then, once constructed, no set may occur under use, or conditions would not be maintained as supposed; and last, but not least, changes in temperature are not allowable since they would greatly change the stresses. Thus, with $E = 30\,000\,000$ lbs. per square inch and $\sigma = 10\,000$ lbs. per square inch, a change of 0.064 ft. in the span would cause a stress of P in the bar z, and the corresponding stresses in the remaining bars (see Fig. 3 or Fig. 9). With a coefficient of expansion of $\frac{6\frac{1}{2}}{1\,000\,000}$ per degree Fahrenheit, 100° Fahr. would correspond to a change in span of $\frac{6\frac{1}{2} \times 100 \times 40}{1\,000\,000} = 0.4 \times 0.065$ ft., or $\frac{4}{10}$ P stress in z, nearly. This result is shown in Fig. 15. And while 100° Fahr. change from the mean (that is 200° Fahr. range of temperature), may be excessive, even if we take only half that or 50° Fahr. change (100° Fahr. range), we get temperature stresses of half those shown in Fig. 15 which, by comparison with those in Fig. 13, we see are some 45% of the greatest stress in z, 22% of that in e, 20% of that in c, 11% of that in a, 10% of that in b, and 6% of that in d. That these stresses, absent from the three-hinged arch, turn the balance completely in its favor, many times offsetting the 3½% economy in the two-hinged arch found by neglecting temperature changes, is self-evident to any impartial student of the subject.

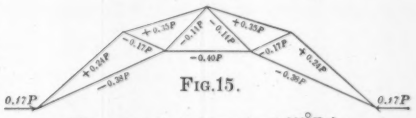


FIG. 15.

Temperature stresses for +100° Fahr.

Finally, we may note that even the 3½% theoretic advantage of the two-hinged arch can be exceeded without the use of indeterminate construction through the use of what has elsewhere been termed "multiple construction." Thus we have the three-hinged arch design

of Fig. 16, capable of supporting all the loadings except $L\,C\,R$ with stress σ or less (see Figs. 4, 5, 6 and 7). For the load $L\,C\,R$ we introduce a bar z of area $\frac{2}{7}\frac{P}{\sigma}$, give the junction of the bars a_l and a_r freedom to move vertically with respect to the junction of the bars b_l and b_r, and load the junction of $a_l a_r$ with $\frac{5}{7}P$, and the junction of $b_l b_r$ with the remainder or $\frac{2}{7}P$. The resulting stresses in this framework, which is statically determined, are shown in Fig. 17. We see that these stresses are equal to or less than the designing stresses shown in Fig. 16. The construction permitting of this might be any such as would ordinarily leave z unstressed, but, as by the insertion of a wedge, cause it to take compression when desired, while the joint of $a_l a_r$ could at the same time be freed from the joint of $b_l b_r$, say by removing another wedge. But it suffices to note that the construction is possible; with the mechanical details we are not here concerned.

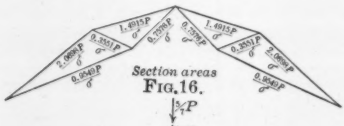


FIG. 16.

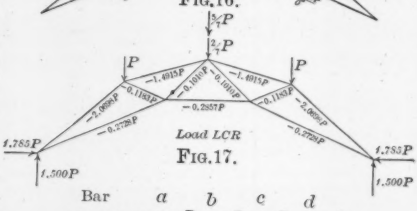


FIG. 17.

Bar	a	b	c	d
Volume	$15.57\frac{P}{\sigma}$	$5.36\frac{P}{\sigma}$	$1.91\frac{P}{\sigma}$	$27.85\frac{P}{\sigma}$
Bar	e	z	Total	
Volume	$15.81\frac{P}{\sigma}$	$2.86\frac{P}{\sigma}$	$135.84\frac{P}{\sigma}$	

The volume of material here, as shown under Fig. 17, is some $1\frac{1}{2}\%$ less than that required by the most economic indeterminate design. Very possibly other multiple determinate designs might be found with still somewhat more economy of material. But with these we need not concern ourselves, the essential point being simply that, as has been stated previously, there will be some statically determined method of solution more economic than any indeterminate solution, even on the theoretic conditions most favorable to the latter. It is not intended to intimate, however, that these multiple determinate solutions are in general desirable substitutes for the simpler but less economic single determinate solutions. The multiple solutions present more or less considerable mechanical difficulties in their adaptation to practical use, and while these difficulties are not

insuperable, in fact, perhaps, are less considerable than the exact construction of an indeterminate framework, they would probably prevent their use where the saving was not considerable. In a case like the present, where the saving would be but some 5%, it is hardly likely that they would be used.

This simple example of the indeterminate arch, although valueless as a design because of the neglect of post formulas, as a comparative study, is affected little by such considerations, and illustrates very well certain points in economic design. While, theoretically, the indeterminate framework may give a more economic design for several loadings, than does any single determinate framework of included figure, the amount of such theoretic advantage is considerable only in most extraordinary cases; and there it would probably pay to use a multiple determinate design which would be still more economic. In all cases where the figure of the simple determinate design closely approximates to the figure of the indeterminate design the theoretic advantage possible through using the latter will certainly be insignificant. Actually, owing to temperature stresses, etc., that advantage will not exist at all. Again, as the structure becomes larger, and the variations of stress due to variable loadings affect smaller and smaller parts of the structure to a sensible degree, the possible economy of indeterminate construction proportionally diminishes. It may safely be assumed that in actual structures (as arches and suspension bridges), a theoretic advantage of even 5% cannot exist in the most economic indeterminate design over the most economic determinate design of included figure, even when temperature stresses are neglected. It is doubtful if ordinarily 1% economy could be found; and temperature stresses alone would offset such advantages several times over in all cases. This illustration, then, definitely confirms the view that the use of indeterminate frameworks is certainly not advantageous from an economic standpoint, nor apparently from any other standpoint.

DISCUSSION.

Mr. Goldmark. HENRY GOLDMARK, M. Am. Soc. C. E. — In this interesting paper, the author applies the well-known principle of Virtual Velocities to a study of the stresses and deflections in trusses with redundant members and other structures not statically determinate.

As the result of his investigation, he arrives at the conclusion that such constructions show no advantage, either in economy of material or in rigidity, over the simpler determinate forms, while even the best methods for computing their stresses are extremely laborious and of doubtful accuracy.

He therefore recommends that all indeterminate forms, such as trusses with a multiple web system, continuous girders, arches with less than three hinges, suspension bridges with continuous stiffening girders, etc., should be abandoned entirely in practice.

As to the method of computation which he has used (that of Virtual Velocities or Least Work), he considers the amount of numerical labor involved so great as to make the method practically valueless.

While the speaker is by no means an advocate of complex truss forms where there is no good reason for using them, he is fully satisfied that there are many cases in practice where these tabooed forms give by far the best results.

As a matter of fact, the simplicity of the strain sheet is, in many cases, by no means a proper criterion as to the excellence of the design. Thus, in railroad bridges of moderate length, the tendency of late years, as the result of actual experience, is away from the "ideal" single-intersection bridge with alleged frictionless pins. Such spans are now almost universally built with riveted connections, and, in many cases, with multiple systems of webbing. Even in larger bridges, the floor system is almost invariably riveted between the posts, though the transference of loads to the main trusses is thus made less direct. In all these cases intricate secondary stresses are introduced, but the advantages obtained are believed to counterbalance this theoretical drawback.

In the case of the larger arches and suspension bridges the introduction of hinges at the center is properly considered to break the continuity of the floor system and wind bracing, to add to the complication of the details, and in all probability to decrease the rigidity of the whole bridge under moving loads.

It is true that the absence of hinges makes the computation of stresses more difficult and somewhat less certain, but it is believed that with the homogeneous material now used, and the excellent workmanship of our shops, perfectly safe structures can be built without any

excessive cost. Besides this, there is, in the speaker's mind, little doubt that even the best designed hinges or pins can hardly be counted upon to act as theory requires. Mr. Goldmark.

But besides such bridge forms as those above mentioned, in which the designer is at liberty to choose between so-called determinate and indeterminate forms, there are many engineering structures which are, of necessity, of a complex nature. Such are the frame works for the enclosing of large shops and auditoriums, in which the roof trusses and the steel posts which support them are connected by knee-braces or other connections, and many forms of cranes and derricks; furthermore, metallic gates for canal locks, as well as ship caissons for dry docks and harbor works. In all these constructions some method of computing indeterminate stresses is desirable.

In such cases, as well as for computing bridge stresses, the speaker has found the Method of Virtual Displacements (Method of Least Work) of great value, while he has not found it excessively laborious, particularly if it is used as a means of devising approximate methods of sufficient exactness in each case rather than for computing individual designs.

A brief reference to the bibliography of this method of computation may be of some interest.

The first application of the principle of Virtual Velocities to the determination of deflections and stresses in frameworks is commonly ascribed to Clerk Maxwell, whose paper embodying this method appeared in the *Philosophical Magazine* for 1864. The same theorem, in a slightly different form, was, however, used independently, to solve problems of this class, by M. Menébrea,* in 1858, and Professor Lamé,† in 1866.

The further systematic development of the method is, however, due mainly to German writers. The earlier articles of Professors Mohr‡ and Winkler§ gave an application of the method to a variety of structures, such as continuous girders, trusses with multiple web systems, arches with less than three hinges, etc. The most complete exposition of the more recent German methods, however, is given in the works of Professor H. Müller-Breslau, of Berlin. His special treatise‖ on the subject the speaker has found of great assistance in practice, and he would consider an English translation of much value for American engineers.

Reference should also be made to a monograph¶ in French, by the late M. Castigliano, a young Italian engineer of great promise, who

* *Comptes Rendus*, 1858, Vol. xlvi, p. 1056.

† "Leçons sur la Théorie Mathematique d'Elasticité des Corps Solides."

‡ *Zeitschrift des Architekten und Ingenieur Vereins zu Hannover*, 1874 and 1875.

§ *Zeitschrift des Architekten und Ingenieur Vereins zu Hannover*, 1879.

‖ "Die neueren Methoden der Festigkeitslehre." Berlin.

¶ "Théorie de l'Équilibre des Systèmes Élastiques." Turin, 1879.

Mr. Goldmark. gives a very elegant analysis of the "Method of Least Work," with many applications.

In the English language, the valuable paper by George F. Swain, M. Am. Soc. C. E., published in 1883,* is the only extended article on this subject known to the speaker, though brief expositions are given in some textbooks on applied mechanics.† Mr. Cilley has therefore conferred an obligation, on English-speaking engineers, by his interesting study on the subject.

Mr. Lindenthal. GUSTAV LINDENTHAL, M. Am. Soc. C. E. (by letter).—This paper is a contribution to the old controversy as to whether or not statically determinate structures are superior to statically indeterminate ones, and is a scholarly attempt on the affirmative side of the question.

Mr. Cilley, like others before him, advances as principal reasons in favor of determinate designs, their alleged economy, the greater certainty of their calculation, and the greater accuracy, therefore, in the dimensioning of the cross-sections.

Granting, that the last two arguments have much to commend them, it is nevertheless true, as regards economy, that diminutive differences in weight of metal, as shown by strain sheets of one and another type, will rarely be the sole deciding element in the choice of designs. Other considerations than mere economy of metal influence greatly the cost and merit of a structure. But in any case, strain sheets to be comparable should be complete, and include all strains which affect the sections. This does not seem to the writer to be the case with the strain diagrams in the paper. Before indicating in what the incompleteness is believed to consist, the preliminary question is justified:

"On what theoretically equivalent basis (aside from same length of span and moving load), should a comparison of the economy of determinate with indeterminate arch types be made? Should it be on the basis of the same figure of inclusion, as the author has done, or on the basis of congruous equilibrium polygons under some assumed load, as would undoubtedly be the case for ribbed arches, or for suspension bridges, which are nothing but inverted arches?"

The short span of primitive figure, chosen by the author with only 10, respectively 11, members in the frame, serves to illustrate the great labor of a detailed analysis for each member of the indeterminate arch; but in all other respects it is misleading. If the diagrams of the two types were each enlarged from 4 to, say, 40 panels, then the determinate figure of the author would become the form known as the Eads arch, for which a certain depth at the quarter is economic, and the indeterminate figure would become a sickle arch, for which another depth at the center is economic. The enlarged diagrams, however, would no longer have the same figure of inclusion. Only one chord,

* *Journal*, Franklin Institute, February, March and April, 1883.

† Cotterill, "Applied Mechanics," p. 551.

namely the upper one, would coincide in both, but that is not enough to furnish equivalent conditions of economy. Mr. Lindenthal.

Suppose a comparison were made between single (determinate) and continuous (indeterminate) trusses. Obviously, we would start out with the same ratio of height to span (at center of span) for each kind, and compare the strain sheets and material required for them on that basis, to come to a correct conclusion of their economy. Similarly, a comparison of arches will have to proceed. We cannot compare fairly the economic merits of arches of different rise, or of an arch having a ratio of rise to span of three-tenths with another arch having the same ratio as to one chord only and another much smaller ratio as to the lower chord, as Mr. Cilley does. He compares two primitive figures, one statically determinate, and the other made indeterminate by inserting a redundant member. Considered as generic forms, the figures are dissimilar and not comparable. His conclusions as to the economy, or the want of it, in indeterminate forms, arrived at in this manner, are wholly irrelevant and inapplicable to long spans, the only ones for which arch types are economically used.

This will appear more clearly when we consider that the closed frame of the author, with a depth of one-eighth of the span at the center would in practice occur as a sickle truss which is determinate, but hardly as a sickle arch which is indeterminate, and with that depth of rib, would be wasteful. No arch bridge has been built yet with a depth of rib of more than one-sixteenth. More frequently it is less than one-thirty-second of the span.

As the temperature strains for the same arch form diminish directly with the ratio of rib to span, they are, in existing bridges, very much smaller than those given by Mr. Cilley on page 405. His maximum is $\pm$ 45% of the full load for $\pm$ 100° Fahr. In the largest sickle arches thus far built, namely, that of the Garabit Viaduct in France and of the Gruenenthal Bridge over the Baltic Canal in Germany, the maxima are under $\pm$ 18%, when corrected to the same extremes of temperature.

But, as stated before, a comparison between forms of the general characteristics of sickle arches and Eads arches, simply because they happen to coincide as to one chord, can lead to no rational conclusions.

Comparisons of carefully worked out designs of different arch types, made by him and others, have convinced the writer that, other things being equal, the indeterminate types are invariably more economical. This is true even if the assumption that there are no temperature strains in three-hinged types were correct. But this assumption is not correct as shown by the writer long ago.* That time-honored theory, found in all text books, is based on error. Three-hinged arches are not

* *Engineering News*, 1888, p. 174; and "Appendix to Report of Board of Engineer Officers as to Maximum Span Practicable for Suspension Bridges," 1894, p. 68, and elsewhere.

Mr. Lindenthal. only not free from temperature strains, but for certain conditions such strains are just as large as in arches of the two-hinged type. The difference is merely in their distribution. The supposed advantage in that respect of the three-hinged over the two-hinged arch is illusory.

Professors Merriman and Jacoby's "Higher Structures" is the only work, so far, referring to the writer's theory on this subject. But although mentioned there merely in connection with three-hinged stiffening trusses in suspension bridges, it applies, nevertheless, to all forms of arches with three hinges, whether of the ribbed, spandrel-braced, or of the erect or suspended form. Each is, necessarily, differently affected; but in all cases the strains are due to the changes in the curvature of the equilibrium curve from dead load for different temperatures.

If the author will trace the strains, due to the falling and rising of the center hinge from changes of temperature, he will find them by no means of negligible smallness, even for the large ratio of rise to span in his inconclusive figure.

The temperature strains are particularly large in flat arches, and, to repeat it, the insertion of a center hinge does not eliminate them. It only serves to remove the maximum bending moment from the center to each quarter. So that where the two-hinged arch has one maximum bending moment, the three-hinged has two of the same intensity in most cases.

The new Alexander III Bridge in Paris, for instance, having a rise of only one-seventeenth of the span, is thus far the flattest metal arch attempted. It is of the three-hinged type, here properly chosen because a small depth of rib was necessitated at the center by the prescribed clearance above the river. The writer has not seen a strain sheet of that bridge, but believes that in accordance with the common theory, the arches have been assumed to be free from temperature strains. As a matter of fact, the bending moments at the quarters, from temperature changes, are fully as great as at the center of a two-hinged arch, having the same depth throughout as at the quarters (where the rib is one seventy-second of the span).

If the Washington Bridge in New York (510-ft. span), or the Niagara Falls road bridge (840-ft. span), which are both of the ribbed-arch type, had been built with three hinges instead of only two, the extra metal required to meet the temperature strains would have been the same as for two-hinged arches, with this difference, that the additions to the chord sections would have been largest at the quarters instead of at the center, and the additions to the web members largest at the ends of each half arch, instead of at the ends of the whole arch. In spandrel-braced arches, the center hinge may be of value, as here it really reduces the intensity of temperature strains, but does not eliminate them, by any means.

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The law applies also to stiffened suspension bridges. The well-known Point Bridge at Pittsburg (800-ft. span) has three hinges; the bracing is above the chains. The bridge has been believed to be free from temperature strains, which is not the case. The maxima from temperature are those corresponding to the maxima at the center of a suspended two-hinged arch, having a rise of one-eighth of the span, and a depth of rib of one thirty-second of the span. But in this case the maxima in the chords, which in the two-hinged arch occur only at the middle of the span, reach, with little variation, from hinge to hinge, while the web is only slightly affected. On the whole, this type, and the so-called Fidler type, are the worst forms for temperature strains, and yet both are determinate forms. The suspended end spans of the Tower Bridge in London, which are of the Fidler type, are certainly exposed to bending strains from temperature changes, although the designers believed them to be eliminated by the insertion of middle hinges.

That the neglect to provide for temperature strains in three-hinged arches has not yet endangered such bridges proves anew how useful the so-called factor of safety is, and how true its designation as a factor of ignorance.

A second correction in the strain sheets, for a proper comparison of the two types treated by Mr. Cilley, is required for the large bending moments resulting from the immobility of the hinges, incorrectly ignored by him. In none of the existing arch bridges has a turning motion at the hinge ever been observed. The friction is too great. But the motion is not even desirable. There could be no motion without wear, which after a while might affect the strains more than the immobility of the hinges.

The bending moments from that cause cannot be regarded as mere secondary strains of local effect, because they make themselves felt throughout the entire arch frame, and in flat arches or shallow ribs require considerable additions to the sections. It is true that the moments can be much reduced by using for the hinges small and long pins rather than short and large ones, for affording the necessary bearing area. But in any event there are, under equivalent conditions, twice as many bending moments from that cause in three-hinged as in two-hinged arches, while they are absent entirely in hingeless arches, in which the temperature strains are largest.

The present theory of metal arches requires, therefore, the following important corrections:

Three-hinged arches: Present theory, statically determinate system, no temperature strains.—Corrected theory, statically determinate system, temperature strains plus bending strains from three hinges.

Two-hinged arches: Present theory, singly indeterminate system, plus temperature strains.—Corrected theory, singly indeterminate

Mr. Lindenthal. system plus temperature strains, plus bending strains from two hinges.

Hingeless arches: Present theory, two-fold indeterminate system, plus temperature strains; requires, of course, no correction, inasmuch as there are no hinges.

Strain sheets prepared on the corrected theory, for the same rise of equilibrium curve for similar loads in all three types, are alone fairly comparable as to economy of metal. The three-hinged and two-hinged types require additions to the cross-sections of the members over those obtained on present theory. These additions cannot be ignored. When made, the two-hinged arch will in every case show great economy over the three-hinged arch.

The secondary stresses, which occur at the panel points from the elastic deformation of the frame, affect in practice only the details and strength of connections. They are not different from those in trusses, cantilevers, and other frames. They may be ignored in a comparison of strain sheets.

The objection to indeterminate structures cannot very well be based on the ground that the strains cannot be determined with the same exactness as in statically determinate structures, because such is not necessary, either for safety or economy.

The dangerous strains, their limits, and the conditions under which they occur, can always be ascertained with sufficient accuracy without the endless drudgery which, as Mr. Cilley shows, a detailed analysis for each member of a large structure would involve. There is a limit to the refinement of the engineer's calculations, beyond which nothing of practical value can be gained.

With the utmost accuracy of computation, in statically determined or undetermined structures, we will never be able to dispense with a three to ten-fold safety, as the case may be, to cover defects of manufacture, want of uniformity in the material, and the many petty indeterminate, as well as undetermined, stresses which are always present. The minimum and maximum limits of stresses ascertained, their frequency properly judged and provided for, we need have no anxiety for the safety of statically indeterminate structures.

No failures of bridges, caused by their indeterminateness, have yet occurred. Some, over 40 years old, are still carrying safely railroad loads, which, in the meantime, have largely increased. The greater diffusion of the shearing strains, particularly characteristic of indeterminate structures with multiple systems of web members, has prolonged their life. Repairs in a few cases are known to have been necessitated by poor details. Many statically determinate structures, built at the same time, and proportioned for the same unit stresses, have shown less resistance to wear. Some American railroads have already the third generation of metal bridges.

Mr. Lindenthal.

It is not irrelevant to the subject under discussion to refer here to the wide-spread bias against the very useful continuous girder, which is alleged not to be economical. This is true only when the chord members, subject to alternate stresses far within the elastic limit, are required to be largely increased in accordance with the discredited Launhardt formulas, or with other similar rules, unjustified by sound reasoning. We have no proof whatever of the strength of iron or steel being affected by alternating stresses within half the elastic limit. The increase of section, as required by bridge specifications in vogue, for members, subject to such strains, is a waste of money and material. The good results with the very economical, durable and rigid, continuous girders, proportioned and built before the present rules of dimensioning were known, ought to have weight with thinking engineers.

That the writer is not alone in his condemnation of the modern rules for alternate stresses in bridge construction, is gratifyingly evident from the report of the discussion at the International Railway Congress held in London in 1895.*

"But," quoteth the theoretician, "consider the dangerous strains in continuous girders, if a pier should settle, or when iron towers expand in height!" The answer is, that piers should be built so that they will not settle more than a certain allowable amount, and that the variations in the levels on iron towers can be allowed for readily in the dimensioning of the girders. This is a part of engineering science, and surely what the Chaldean and Roman bridge builders accomplished in the way of good foundations, the modern engineer should at least be able to equal. And when secure foundations cannot be had, neither continuous girders nor other forms of indeterminate structures should be chosen. It is the engineer's business to investigate and to discriminate.

The shaky and vibrating cantilever structure, so much affected as a great improvement in that respect, is by no means an adequate substitute for the rigid, compact and economical continuous girder, properly designed.

Another point deserves attention. The author seems to ascribe much importance to deflections as a means of judging the rigidity of a bridge. What is a rigid bridge? If we take rigidity in metallic bridges to mean absence of vibration, then deflections are a deceptive criterion.

The public considers a bridge which vibrates, as an inferior structure; and one which does not, as a superior one. This ought to be the guide, also, to the engineer. Sometimes, structures regarded as rigid have great deflections and little vibration, and *vice versa*, of which the following are a few instances:

The old Niagara single-track railroad suspension bridge (indeter-

* Congress Bulletin, page 79.

Mr. Lindenthal. minate) deflected ordinarily 10 ins., under a train, but a pedestrian on the roadway below would hardly notice it and would feel very little vibration. It was a more rigid structure than the double-track cantilever railroad bridge (determinate) located near by, which deflects ordinarily only 3 ins., but a pedestrian on it feels disagreeable sensations, and can hardly keep his feet when a train passes over the bridge.

The St. Louis arch bridge (indeterminate) vibrates very noticeably under every train and team, while the Merchant's Bridge (determinate) located near by is a fairly rigid truss structure.

The unsightly Market Street cantilever bridge (determinate) in Philadelphia is unpleasantly known for its spring-board motion, while the old cast-iron arc bridge (indeterminate), located below, at Chestnut Street, is rigid. One of the abutments of the latter bridge has yielded about 3 ins., but none of the calamities predicted in such cases for indeterminate arches have occurred. The behavior of three famous steel-arch bridges is also instructive, viz.: The Washington arch-bridge (indeterminate), in New York, has ribs with solid webs; its deflections from ordinary loads are nil, but its vibrations, even under a single-horse truck, are so noticeable that they have been the subject of (of course unfounded) anxious communications to newspapers. If it had three hinges instead of two, the vibrations would be worse. The Niagara Falls ribbed arch bridge (indeterminate), completed two years ago, is similarly complained of, while the spandrel-braced railroad-arch bridge (indeterminate) below is one of the most rigid metal bridges in existence.

Specially interesting, in that respect, are also four different types of large bridges at Buda Pest.

The railroad bridge with continuous lattice girders (indeterminate) is rigid under fast trains; the famous Buda Pest suspension bridge, with no stiffening to speak of (and therefore determinate), shows only little less vibration than the heavy Margarethen spandrel-braced, arch bridge (indeterminate), or the new cantilever bridge (determinate) near by.

Owing to the prejudice against indeterminate structures, most of the very high, iron-trestle viaducts in this country are subject to so much vibration, that trains must greatly reduce speed over them. With girders cut over every post, and provided with slide bearings, as demanded by the usual bridge specifications, sufficient rigidity cannot be obtained. The high viaducts, with continuous girders (indeterminate), in Europe, some of them over 40 years old, are, on the other hand, fairly rigid under fast trains.

Compare the Forth Bridge (indeterminate), proportioned for a live load of 4 000 lbs. per lineal foot, a riveted cantilever structure, in which 15% of the metal is in the strong lateral bracing, with the Brooklyn

Suspension Bridge (indeterminate), proportioned for only 2 000 lbs., having no lateral bracing to speak of, and a very defective stiffening system. The latter bridge is subject to much greater deflection than the Forth Bridge, but both bridges are equally free from noticeable vibration under the loads and at the speeds for which they are designed. Mr. Lindenthal.

The remarkable rigidity of even scantily stiffened suspension bridges and the reasons therefor have been discussed by the writer on former occasions. It is his opinion, that for long spans properly stiffened suspension bridges are the most rigid of any metal type.

Thus the study of existing bridges, of both the determinate and indeterminate kind, affords better instruction in rigidity than the mere comparison of deflections. Naturally, these ought to be always as small as possible.

The writer does not advocate indeterminate structures, but neither is he prejudiced against them. A decision as to their true economy and merits can be reached only from case to case. It seems to him that the sweeping conclusions of Mr. Cilley and of others against them are founded on incomplete investigations and are at variance with known facts.

Prof. C. W. RITTER* (by letter).—In his detailed comparison of statically determinate and indeterminate frameworks, the author endeavors to demonstrate the decided superiority of the determinate forms. He claims that the determinate structures are, theoretically, more economical than the indeterminate ones. He shows that in indeterminate structures, a slight inaccuracy in the length of a member or a slight change of temperature, can effect a considerable and even serious change in the stresses. He emphasizes the fact that discrepancies in the levels of the supports of continuous girders, or a slight yield of piers or abutments of two-hinged and hingeless arches, have considerable effect on the interior stresses. He also points out the laboriousness of the exact design of indeterminate structures. Mr. Ritter.

It will scarcely be necessary to concede that all these arguments are doubtless very reasonable, and the writer thinks it wise to emphasize now and then the advantages of determinate structures and to expound the uncertainties of the basis on which the design of indeterminate structures is founded.

In German literature, these questions have often been discussed and analyzed. A thorough investigation of the matter has not been without conclusive results. The use of the multiple intersection system has commonly been dropped; arches which, formerly, were all designed without hinges are now very often provided with two or three of them. Within the last ten years, even several stone arches have been built

* Professor of Civil Engineering, Federal Swiss Polytechnic, Zurich.

Mr. Ritter. with three hinges. The cantilever system has been frequently substituted for the system of continuous girders.

Although a noticeable change has taken place in the manner of constructing frameworks, European engineers have not, as yet, abandoned indeterminate forms entirely, and the writer dares to say they never will. Without disputing the advantages of the determinate forms, a thorough and careful examination of the question leads to the result that, considered from the practical standpoint, the indeterminate frameworks must frequently be declared preferable, at least for European practice.

First, it may easily be perceived that some of the above-mentioned disadvantages of the indeterminate structures are, in many cases, of little weight, and that some others adhere also to the determinate forms. If the foundations are absolutely solid, why should we not build a continuous girder, as well as a cantilever. The influence of varying temperature on two-hinged and hingeless arches decreases with increasing height for the same span, and, in case such arches have other advantages besides, we should not hesitate to prefer them to the three-hinged arch. Unequal warming of the members of an indeterminate framework exerts indeed a disadvantageous influence on the stresses, but the same may be said of the one-sided warming of the members of a determinate structure. In fact, we will never succeed in avoiding entirely the influence of varying temperature.

In regard to economy, the author asserts that the determinate structures need less material than the indeterminate ones. For this assertion he gives a plain and convincing mathematical proof. But let us be careful in applying this result to our practical constructions.

For instance, it can be shown that the framework designated *a*, Fig. 18, which is determinate, requires more material than the framework designated *b*, which is indeterminate. Assuming each panel load to be equal to 4, and that, by introducing initial stresses, the diagonals in each panel in form *b* are equally strained, the stresses indicated by the figures will result. Multiplying these stresses by the lengths of the respective members, we have:

Form *a*—Upper chord	$= 2\,(10 + 16 + 18)$		$= 88$	248
Lower chord	$= 2\,(10 + 16)$		$= 52$	
Posts	$= 10 + 6 + 4 + 6 + 10$		$= 36$	
Diagonals	$= 2\,(14.14 + 8.49 + 2.83)$	$\sqrt{2}$	$= 72$	
Form *b*—Chords	$= 2\,(10 + 13 + 17 + 13 + 17)$		$=140$	232
Posts	$= 7 + 2 + 2 + 2 + 7$		$= 20$	
Diagonals	$= 2\,[14.14 + (2\times 4.24) + (2\times 1.41)]$	$\sqrt{2}$	$= 72$	

Nevertheless, the assertion of the author is correct, inasmuch as there exists a third form c, included in b, which requires still less material, for we have: Mr. Ritter.

Form c—Upper chord	$= 2\,(10 + 12 + 16)$	$= 76$	224
Lower chord	$= 2\,(14 + 18)$	$= 64$	
Posts	$= 6 + 6$	$= 12$	
Diagonals	$= 2\,(14.14 + 5.66 + 2.83 + 2.83)\sqrt{2}$	$= 72$	

At the same time, we recognize that the inner posts have disappeared. As these posts, as a rule, are necessary for attaching floor-

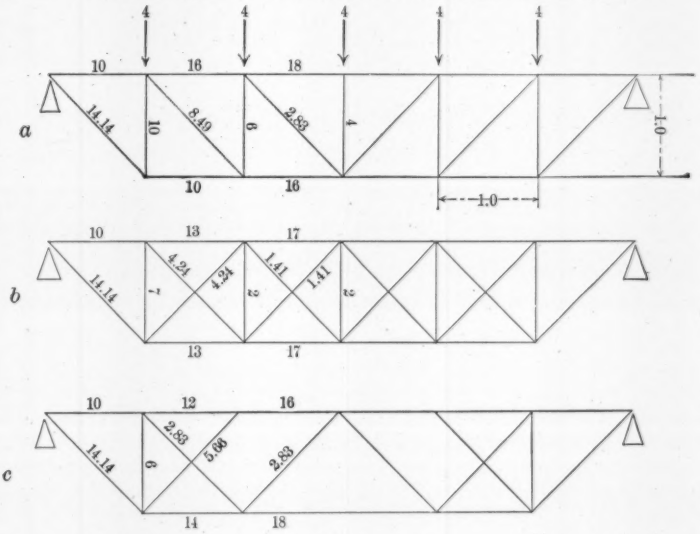

FIG. 18.

beams, the form c is in most cases practically useless, which proves that the advantage of the determinate frameworks, as far as the theoretically required amount of material is concerned, may, by accessory circumstances, easily be reversed into a disadvantage.

The author himself shows that for varying loads the above-mentioned rule loses its validity. There are still other facts which often prove the indeterminate structures to be more economical.

First, in using intersecting diagonals, we obtain the advantage that in these members the stresses are halved, and, in consequence, the struts can be attached directly to the chords; while, in the contrary case, we are obliged to use costly connection plates. Again, by intersecting the ties with the struts, we shorten the distance of supports

Mr. Ritter. of the latter. Further, it is a common experience, that the stresses in the various members of an indeterminate framework are of greater uniformity than those of a determinate one, a fact which naturally leads to some economy. Finally, for cantilevers and three-hinged arches, the cost of the structure is increased by that for the hinges, and it is not unusual that this additional cost exceeds all economy secured otherwise through statical determination. It may be remarked also that the connections of the lateral system, on account of its continuity past the hinges, very often lead to clumsy details and difficulty in construction.

After all, it will be difficult or even impossible to predict which of the two systems will prove cheaper in the end, without computing and designing, in each special case, both of them. The greater labor involved in computing statically indeterminate structures can rarely be a reason for abandoning them. The methods for their computation have been perfected to such a degree that the time required for it alone is hardly of any moment in comparison with that required for the entire design.

One of the chief objections, emphasized by the author, to statically indeterminate structures is the possible discrepancies in the length of members. This objection is an important one for the United States of North America, where pin-connected structures are generally built. It is of less importance in Europe, where riveted connections are used more generally. If the length of any member of an indeterminate, pin-connected framework is inaccurate, the defect, as a rule, cannot be corrected easily, while in riveted connections such deficiencies are compensated by the red-hot rivets which will fill out the holes even if these, within certain limits, do not match. European specifications, not unlike American ones, strictly provide that if, in connecting parts or members, the rivet holes fit badly, the holes have to be enlarged and larger rivets have to be used. Stretching the bars, in order to match the rivet holes, is strictly forbidden. If in spite of these precautions a slight discrepancy remains, it is, as mentioned above, compensated by the rivets themselves, which, if properly driven, will fill out the holes perfectly. It seems to the writer that for this reason the inducement for avoiding statically indeterminate structures exists far less for rigidly rivet-connected than for pin-connected structures.

Regarding stiffness, the author gives us an interesting comparison between the deflection of a two-hinged and a three-hinged arch. He finds that the difference is very insignificant. It seems to the writer that this result, for the example in question, has to be attributed to the fact that the theoretical height of the two-hinged arch is actually less than that of the three-hinged one. In general, structures with hinges (arches or cantilevers) are less stiff than those without hinges. For instance, the fixed span of a cantilever deflects exactly as much as a

single-span girder, while the deflection of a continuous girder of the same span is from 30 to 50% less. Let it be remembered that in railway bridges such hinges often produce shocks, which benefit neither the bridge nor the rolling stock. Mr. Ritter.

In reference to the stiffness of bridges, we should not only consider the elastic deflections under varying loads, but also the vibrations of the structure in a vertical and lateral sense. It can scarcely be contended that, in this respect, statically indeterminate structures are safer, although, as the author says, a strict proof for this assertion can scarcely be given. Now, is not the extensive adherence of American engineers to stiff joints in upper chords, as well as the rigid attachment of floor-beams, stringers, and lateral, portal and sway-bracing, a proof of their sound constructing sense, although all these construction details clearly involve statical indetermination? The author, it is true, refers principally to vertically loaded frameworks, but what is sound for floor beams, stringers and bracing, is not likely to be wrong for frameworks throughout.

The author should be thanked for his intelligent investigations and his manifold suggestions. In any case, it will be unreasonable to build an indeterminate framework where a determinate one is just as advantageous, and the author's endeavor to substitute determinate for indeterminate forms is praiseworthy. But on the other hand, it is the writer's opinion that indeterminate structures are in many cases preferable, and that we will never be able to do without them entirely. Their frequently greater economy and their greater stiffness, as well as their adaptability to various circumstances, are such important advantages, that it would not be wise to throw them overboard on mere principle. On the contrary, it should be regarded as the task of engineering science to study the qualities, the peculiarities and especially the deficiencies of indeterminate structures, to search for necessary remedies, and to simplify further methods of computation.

Prof. W. Dietz* (by letter†).—The writer was a pupil of Head-Master H. Gerber, who, by the introduction of continuous girders, such as cantilevers and those simply supported at the piers, has gained great distinction in the art of bridge building, and who, as is perhaps not generally known, by the construction of the street bridges over the Main, near Hassfurt,‡ furnished the prototype for the truss system of the Firth of Forth Bridge. The writer, therefore, has had the opportunity of becoming intimately acquainted, both theoretically and constructively, with the greatest variety of statically determined truss systems. It follows, therefore, that he has retained a certain preference for such systems, with their strains so clearly and quickly Mr. Dietz.

* Technical Highschool, Munich.

† Translated.

‡ *Zeitschrift für Architektur und Ingenieurwesen*, 1898, p. 563, Fig. 1.

Mr. Dietz. obtained. Yet his vocation makes it necessary for him also to investigate, thoroughly, both theoretically and practically, statically indeterminate systems of the most different kinds, and his experience gained thereby, together with the indisputable advantages of undetermined systems under certain conditions, compel him not to withhold due acknowledgment thereof, and to protest against unfounded attacks upon the existence of these trusses.

The writer, with a large amount of material at his dispcsal, must characterize the author's closing sentence "that the use of indeterminate framework is certainly not advantageous from an economic standpoint, nor apparently from any other standpoint," as not in accordance with facts, and hardly capable of demonstration.

The matter is not as simple as the author represents it to be. As is well known, it is not only the length of span between supports that influences the volume of iron in bridge trusses, but also the greater or smaller number of panels into which the span is divided, the length and form of cross-section of the compression members; the kind of joints, and the manner in which they are made, as also the riveting of adjoining parts; and, finally, to a very great extent, the method used in dimensioning. As regards the latter, the method by Gerber, as specified in Bavaria, which takes into account the impact of variable loading, gives considerably different cross-sections in individual members from those determined by the widely used Launhardt-Weyrauch formula, which does not consider the effect of impact.

In order, therefore, to make a professional comparison, in regard to the amount of iron needed, between several different truss systems, it is necessary to take account of all the foregoing factors, and to observe the condition that all the trusses are dimensioned by the same method.

Most of these factors are taken account of by introducing the so-called construction coefficient with which the theoretical volume of the different members must be multiplied in order to obtain the practical volume. The determination of these coefficients is, however, a work of considerable scope, and leads to results of practical value only when based upon a large number of data, which the bridge companies, for reasons which are obvious, do not give out generally.

Further, it should not be overlooked that the superstructure of a bridge forms only a part thereof, and that not only the metal construction, but also the abutments and piers, the method of erection and the maintenance of the structure, should be taken into consideration, so that in special cases it is quite possible that a considerable increase in the cost of the iron structure may result in a saving on the final cost of the whole bridge. The ultimate object of all our technical information and knowledge is to arrive at the lowest possible total cost, providing at the same time for an even strength of all the parts.

Mr. Dietz.

There is ample proof to show how often the effort is made in Germany, even in extraordinary cases, to use statically determined truss systems, as, for instance, in the Neckar Bridge, in Mannheim* and the Donau Bridge in Budapest.† If, however, in a large number of structures‡, built of late, indeterminate frameworks have been used in the main truss systems, the reason for this should not be sought in a one-sided preference for difficult theoretical problems, but in the fact that after a careful consideration of all circumstances in each case, these systems were found to be the most advantageous.

As a particularly instructive example of this class, the Müngsten Viaduct§ can be mentioned. In this case the most thorough comparative studies regarding the effects of brakes, and the matter of erection, led finally to the selection of a combination of trestle bridge with a statically indeterminate arch with fixed ends.

It should be noted, in this connection, that very interesting experiments were made upon this bridge, and that they showed a very satisfactory agreement between the theoretical and the actual strains in the truss members.

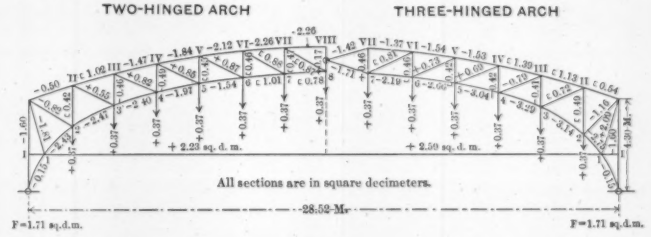

FIG. 19.

As is well known, the Germans are much interested in the thorough investigation of the so-called secondary strains‖ which are found in every truss of whatever system, arising either from the inflexible, riveted joints, or from the frictional resistance of the pins, which, even in the best constructed, pin-connected bridges must be overcome before an actual turning of the individual members about the pin can take place. From this point of view the two-hinged arch, for instance, is, when properly designed, considerably superior to the ordinary truss bridge resting on two supports; and this is an additional factor which in connection with a pleasing outer appearance,

* *Zeitschrift des Vereins Deutscher Ingenieure*, 1891, p. 89, Fig. 12, and *Zeitschrift für Architektur und Ingenieurwesen*, 1898, Plate xiv, Fig. 11.

† *Zeitschrift des Vereins Deutscher Ingenieure*, 1894. International Competition, p. 1238, Fig. 72.

‡ *Zeitschrift für Architekture und Ingenieurwesen*, 1898, p. 561.

§ *Zeitschrift des Vereins Deutscher Ingenieure*. "The Müngsten Viaduct." p. 1321.

‖ Engesser: "Supplementary Forces and Secondary Strains in Iron Truss-Bridges," 1892 and 1893.

Mr. Dietz. which the Germans happily are appreciating more and more, has influence in determining the choice of a truss system.

In the preceding, the writer has outlined the different elements which should be considered in judging the merits of different truss systems; he would now, in conclusion, prove by figures how little permissible it is to draw general conclusions as to the economical superiority of the three-hinged over the two-hinged arch, even in purely theoretical respects, from the author's comparisons between quite small arches of definite shape.

The diagram, Fig. 19, illustrates the very carefully made comparisons between a three-hinged and a two-hinged arch with elastic tie rod for the Hacker Bridge in Munich.* In the right half the theoretically determined cross-sections for a three-hinged arch are given, and in the left half those for a two-hinged arch, in square decimeters. Pure tensile strains are indicated by +; pure compressive strains by —, and members subject to both kinds of strain by [.

The result of this comparison is very unfavorable to the three-hinged arch, even when only the theoretically determined volumes are considered, inasmuch as it requires 11.3% more material than the two-hinged arch, and this difference becomes still greater when the additional material required for the center hinge is taken into account. A still greater disadvantage for the three-hinged arch appears when its deflection is considered, as it amounts to 30.5% more than that of the two-hinged; it was calculated carefully for both arches, according to the well-known method by Mohr, taking into account the change of length in every member.

The writer maintains that the question, whether under all circumstances the statically determinate framework is superior to the indeterminate, cannot at all be answered on the basis of a one-sided comparison, considering only the theoretically required amount of material. It can only be determined in each case by weighing carefully the influences of all the factors, and requires, not only thorough theoretical knowledge, but also, a large amount of comparative material and experience.

Mr. Sohn. Prof. JOSEPH SOHN (by letter).—The exact design of statically indeterminate frameworks, as explained in the paper, is based essentially on the idea that the stresses of the superfluous bars under a certain loading are not computed but assumed. In the case of a single given loading (constant loading), the author assumes arbitrarily the stresses in the superfluous bars, deduces thence and from the loading the stresses in the remaining (necessary) bars and designs on this basis the cross-sections of the various members. The equations of elasticity determine then the differences between the figure length and the actual un-

* *Zeitschrift des Vereins Deutscher Ingenieure*, 1893. "Construction of the Hacker Bridge" p. 1441.

strained length of each superfluous bar. Thereby, the construction is designed, and in practice it only remains to give to the bars the corresponding length. The more nearly this condition is attained the more nearly the actual stresses approach the designed. Mr. Sohn.

If a statically indeterminate framework is subject to the several different loadings (variable loading), the author assumes the stresses in the superfluous bars under one of the given loadings or, more generally, he sets an equal number of further conditions; in the equations of elasticity, each relating to the difference between the stresses in the bars under two of the given loadings, and therefore not containing the differences between the figure and actual unstrained length of the superfluous bars, the author expresses the section areas of every bar by the corresponding stress under the most unfavorable loading; the equations of elasticity determine, then, the stresses in the superfluous bars under each of the given loadings; thence result the stresses in the necessary bars, and it remains to compute the differences between the figure length and actual unstrained length of the superfluous bars.

The case of a constant loading offers, in designing, no difficulty; with variable loading, on the contrary, the design, even under the simplest conditions, is exceedingly complicated and laborious. The author himself accents it expressly and adds force principally upon the proof that statically indeterminate systems are in all respects (economy, stiffness, safety) inferior to the determinate constructions, and that only these last systems may be recommended in practice.

Although there are cases where it will not do to exclude statically indeterminate forms, the writer agrees with the author that statically determinate systems generally are to be preferred, particularly for the reason that slight imperfections of the actual construction, unavoidable in practice, do not influence essentially the stresses in the bars. In statically indeterminate constructions that influence is, on the contrary, very considerable; wherefore important divergences between the computed and actual intensities of stress are possible and probable. A simple example, given by the author, shows evidently how much the intensity of stress can be altered by the incorrect length of a bar.

G. Jung,* Esq. (by letter†).—The writer has read Mr. Cilley's paper on indeterminate frameworks with much interest and takes pleasure in finding that he entirely agrees with him. Mr. Jung.

For a long time the writer has maintained that statically determinate frameworks are preferable to indeterminate ones. His reasons for this opinion being about the same as those which the author has so well set forth, he considers it useless to repeat them here.

On the other hand, he has for many years, in his course on graphical

* Associate Manager of the *Annali di Matematica*, Milan, Italy.

† Translated.

Mr. Jung. statics at the Superior Technical Institute of Milan, called the attention of students to the paper of Maurice Levy (mentioned also by Mr. Cilley), and has invited them to study the argument treated therein. With considerations of a general character, analogous to those given on pages 570 and 571 of the paper, the writer emphasizes the simplicity of the calculation and of the hypotheses required by statically determinate frameworks as compared with the complicated calculations and the necessity of further hypotheses, required by indeterminate frameworks, and concludes with pointing out the superiority of the former structures over the latter, at least from the theoretical and purely statical (and economical) point of view.

As for the dynamical phenomena, the writer does not know of any; but the author's observations on page 361 seem to be just. The writer does not know, either, what the good reasons are which could make one prefer indeterminate rather than statically determinate frameworks, which latter have in their favor, besides, the practice and the large experience of American engineers.

Mr. Cilley. FRANK H. CILLEY, S. B. (by letter).—The writer believes that Mr. Goldmark's objections rest, in part, on a misapprehension. The use of methods of calculation based on the principle of virtual "Velocities" or "Displacements" or "Work," as it is variously termed, or which result from the principle of least work, which is simply an integral form of the preceding, is the simplest as well as the most correct means of determining changes in stress and strain in indeterminate structures. Where such structures exist, and, therefore, must be examined, the writer most strongly approves the use of these methods. And in designing, if engineers will use indeterminate forms, these methods are still to be preferred. But in this case the present tentative method of application must be used, as the direct and exact method outlined by the writer is too difficult to be practicable. The writer's point is that while the methods just referred to are the best known, still, being indirect and tentative, and embodying no principle that guides to good rather than bad proportions, even their use in designing is necessarily unsatisfactory, and, consequently, designs depending on their use will be correspondingly defective. The writer's objections are directed rather to the unnecessary use of systems of construction involving these uncertain, wearisome and unsatisfactory calculations than to the methods of making these calculations.

Regarding the alleged tendency to multiple systems of webbing, the writer would refer Mr. Goldmark to Professor Ritter's statement in his contribution to this discussion, that "the use of the multiple intersection system has commonly been dropped" as a result of German investigations of the subject.

Mr. Goldmark's references to the bibliography of the subject are

valuable. The writer personally prefers the use of the principle of "Virtual Velocities" (displacements or work), so clearly set forth in Professor Swain's article, "On the Application of the Principle of Virtual Velocities to the Determination of the Deflection and Stresses in Frames," elsewhere referred to, to the use of the integral principle of "Least Work," of which an excellent exposition by Professor William Cain will be found in the *Transactions** of this Society. The former principle, while leading to precisely the same equations as the latter, is much more objective, and, in the writer's opinion, is less likely to introduce errors. Mr. Cilley.

Mr. Lindenthal's objections raise many most interesting points which the writer only fears his present very limited time will not permit him to discuss with the thoroughness which is desirable. He trusts that a further opportunity will permit of his correcting present shortcomings, and that brevity will be pardoned in this instance.

One of Mr. Lindenthal's first objections is to the comparison of a given indeterminate framework with a determinate framework of included figure, a procedure which he characterizes as unfair. In this the writer will agree with him, except that the unfairness is not to the indeterminate, but to the determinate form. For the indeterminate form is any given form, and, certainly, it would be fair in theory and always permissible in practice to use in its place another framework of included figure. It rises no higher, it descends no lower, it extends in nowise beyond the bounds of the given framework, and offers at least as great clearances in all directions. What just objection, then, can be made to its substitution for the given framework? And if it is true that this leads to a greater "effective" height or depth in the substituted framework, that is only one of the legitimate advantages of the substitution. It is in part because of such advantages that the determinate forms are gainers over the indeterminate. But the comparison, even so, is unfair to the determinate framework, for all the proportions of the indeterminate framework may be those which most favor it, while the determinate framework, whose figure is formed simply by dropping out certain lines of the figure of the given indeterminate framework, without any alteration in proportions, may not, and very likely will not, be the best determinate framework whose figure simply does not exceed the bounds of the figure of the given indeterminate framework. This latter would certainly be a fairer basis, but even that would not be wholly fair. Usually, bounds in many places exceeding those of the given indeterminate framework would be equally permissible. It would only be fair to permit the search for a superior determinate framework to extend to any within these bounds. It must be remembered that the indeterminate framework is supposed to be given and the question raised is: Can a determin-

* *Transactions*, Am. Soc. C. E., Vol. xxiv, p. 265.

Mr. Cilley. ate framework be found which would equally well satisfy the actual limiting conditions, and which would advantageously replace the given framework? Could this be shown in general to have an affirmative answer, the superiority of determinate over indeterminate construction would thereby be demonstrated conclusively.

But it is not possible to thus broaden the limits in a general discussion. We must remain within limits which are beyond question, and those surely are the boundaries of the given indeterminate framework. The writer has even narrowed these limits still further, and required the bars of the determinate framework to occupy no other positions than those of corresponding bars in the given indeterminate framework. As a result, while the demonstration that there exists a superior determinate framework under these conditions is absolutely conclusive, the demonstration of the contrary would still leave open the possibility of a reversal through fairer conditions.

Mr. Lindenthal's next point is an objection directed to the very considerable proportion of depth to span in the arch of the second illustration. It is one-eighth, whereas in practice one-sixteenth is about the maximum and one thirty-second more frequent. Now, this arch was proportioned as a two-hinged and not as a three-hinged arch. The small number of its members (required to reduce the work of an "exact design") alone is responsible for this considerable depth. The three-hinged design obtained by dropping out the middle bar of the two-hinged design is far from being of a superior character. For the limited number of members used, the three-hinged design is by no means favored in its proportions. If it gains decidedly in effective arch rise at the center through this great depth of the two-hinged arch, it may be contended, on the other hand, that the great depth of the two-hinged arch at the middle should secure it great rigidity. We should expect that where the characteristic difference of two types was most marked, there we would find the most marked expression of the superiority of one or the other. And surely it is the absence of all stiffness at the center of the three-hinged arch, and the considerable stiffness at the center of the two-hinged arch, which are the most characteristic contrasting features of the two. If, where these are most marked, we find economic and other differences small, how much less will they become as these differences in character become less marked. It must be remembered that the writer is contending that decided advantages in economy, stiffness and safety are not obtainable through the use of indeterminate frameworks, and, therefore, that other considerations favoring determinate construction decidedly outweigh any small advantages in these directions which indeterminate structures may occasionally possess. It is, therefore, a very important point to have shown, as in the illustration to the paper, that even in an extreme case, where large divergence was to

be expected, actually, it was small. Therein, in part, is the justification for the statement that in actual and usual cases the differences will be much smaller still. Mr. Cilley.

We now come to another point, and this is a very important one, in the comparison of determinate and indeterminate forms—the question of flexibility. If a structure be very flexible, that is, permit of relatively very considerable changes of form, if it is an indeterminate structure, it is evident that the importance of adverse factors, such as temperature changes, yielding supports and inaccurate construction, will be greatly lessened. That is to say, in slender structures the defects of indetermination tend to become negligible. Now, an arch with a rib depth less than one thirty-second of the span certainly is a slender and relatively flexible structure, and, if arches are usually of such proportions as Mr. Lindenthal states, then they are usually in the category of structures in which the evil consequences of indetermination are minimized. Ordinary, stiffened suspension bridges are also in this category.

Now, these flexible types really form a class by themselves. Their distortions under load are so considerable that the ordinary theory of rigid structures does not closely apply to them. The terms "statically determinate" and "indeterminate" cease to have their old meanings, and consideration of elastic displacements becomes necessary in all cases. It is true that such consideration is not ordinarily given, but that is chiefly because of our ignorance and its difficulty—not because it is not needed.

The writer's study can make no pretense of including such structures. Based on the application of the principle of virtual displacements, it certainly does not apply when the actual displacements result in very sensible directional changes. And it will be well, at this point, to eliminate from this discussion the discussion of the comparative merits of determination and indetermination in such structures. It is an interesting subject, but occupies a field by itself. Here, we are only concerned with the location of its border line. The writer ventures the temporary suggestion that structures whose changes in form under load result in a change of more than 10% from the stresses which would result if the changes in form were insignificant, be considered as of the class of flexible structures here excluded.

Limiting ourselves, then, to the consideration of non-flexible structures, the writer's statement of the great importance of temperature stresses in indeterminate structures is well founded, and his illustration with the deep arch a proper example. The suspicion is probably justified that, in indeterminate arches in which the temperature stresses have been found to be very small, the distortions are actually very considerable, and the calculated stresses under load, obtained in the usual ways, far from expressing the actual stresses.

Mr. Cilley. Mr. Lindenthal tells us that:

"Comparisons of carefully worked out designs of different arch types, made by him and others, have convinced him that, other things being equal, the indeterminate types are invariably more economical."

Without questioning his sincerity in this statement, it may fairly be asked that he shall produce the data in order that judgment may be passed on the fairness of the comparisons. The writer's experience in these matters is, that really fair comparisons are practically unknown, not because of prejudice on the part of those making the comparisons, but because of the unconscious introduction of unreasonable limitations. Thus, if a three-hinged arch is to be designed for comparison with a two-hinged arch, the middle hinge is usually placed midway between the top and bottom chords, or at the bottom chord, in spite of the obvious fact that the horizonal thrust is diminished by putting the hinge as high as possible; that is to say, in the top chord. Or, if it is a comparison of the three-hinged arch with a hingeless arch, two of the hinges are always placed at the abutments, in spite of the fact that it may be decidedly more advantageous to put them farther out, and that it is legitimate to do this and subject the abutments to a bending moment which they would have to resist in any event with the hingeless arch.

Another form of unfair discrimination is that mentioned by Professor Jacoby,* where he speaks of the designing of a three-hinged arch to compare with a given two-hinged arch, and giving the sections of the lower chord of the former such proportions that, in spite of its being heavier, its radius of gyration averaged 4% less than that of the sections of the lower chord of the two-hinged arch. In spite of this fact, however, in this case the three-hinged design proved slightly the more economical. The designing of the cross-sections is a most important factor in fair comparisons, and it alone, by its variation, may easily throw the balance in favor of one type or the other in the comparison of actual designs. This fact is one of those which render the comparison of actual designs so unreliable a basis for determining the relative economy of types.

We now come to a point on which Mr. Lindenthal correctly insists, but, the writer fears, without due regard to proportion—the fact that changes in temperature modify the stresses, even in many determinate frameworks, contrary to the prevalent idea. But Mr. Lindenthal goes so far as to state that:

"Three-hinged arches are not only not free from temperature strains, but for certain conditions such strains are just as large as in arches of the two-hinged type. * * * The supposed advantage in that respect of the three-hinged over the two-hinged arch is illusory."

While the changes in stress resulting from temperature (which are not precisely temperature stresses) are facts and should be considered,

* *Transactions*, Am. Soc. C. E., Vol. xliii, p. 31.

the writer feels that Mr. Lindenthal's quantitative statements should be accompanied by the data on which they are based so that we may see for what "certain" conditions these stresses accompanying temperature changes are so considerable. Mr. Cilley.

If we look into these so-called temperature stresses of Mr. Lindenthal a little closer we shall note that they are simply a special case of the modification of the stresses in a structure resulting from changes in its form, however brought about. The loadings bring about such changes in form, and, as previously noted, to such an extent in the more flexible structures that the ordinary theory which neglects such changes is seriously in error. Now the fact is simply this, that structures whose figures are altered seriously by temperature changes are likely to have their figures altered even more seriously by their loading; that is to say, are to be classed among the flexible structures mentioned previously. While calling attention to these temperature changes Mr. Lindenthal is neglecting other, even more important, changes which occur in these cases.

And another fact is to be observed: These neglected changes, whether from temperature or load, occur in the indeterminate frameworks as well as in the determinate, and add their effects to one as well as to the other; so that Mr. Lindenthal's neglected factor is one that must be applied, both to the indeterminate and the determinate frameworks. Very flat arches, whether three-hinged, two-hinged or hingeless, suffer serious alteration in their stresses from those ordinarily determined, through their flattening very sensibly further, whether from fall of temperature or from the application of the loads. It is a mistake to suppose that the present usual theory of indeterminate structures takes this flattening into account, whether in connection with temperature or load stresses. Actually, it neglects it as completely as does the ordinary theory of determinate structures. To see this and to enable us to form a better idea of these alterations of stress, we must develop their theory.

Theory of Changes of Stress in Frameworks Due to Changes in Form.

Suppose we denote by P the loads, by S the stresses, by R the reactions, and by α the angles of the bars, loads and reactions with (say) the horizontal, confining ourselves here, for simplicity, to plane frameworks. Then, as we know, on the ordinary supposition of no appreciable change in form of the framework under load for each joint we have equations of the form :

$$\left.\begin{aligned} \Sigma P \cos.\ \alpha + \Sigma R \cos.\ \alpha + \Sigma S \cos.\ \alpha = 0 \\ \Sigma P \sin.\ \alpha + \Sigma R \sin.\ \alpha + \Sigma S \sin.\ \alpha = 0 \end{aligned}\right\} \ldots\ldots\ldots (A)$$

But if the framework actually does change in form and the α

Mr. Cilley. become $\alpha + \Delta_\alpha$, the S become $S + \Delta_S$, the R become $R + \Delta_R$, and the P become $P + \Delta_P$, then our joint equations become:

$$\left.\begin{aligned} \Sigma\,(P + \Delta_P)\cos.\,(\alpha + \Delta_\alpha) + \Sigma\,(R + \Delta_R)\cos.\,(\alpha + \Delta_\alpha) + \\ \Sigma\,(S + \Delta_S)\cos.\,(\alpha + \Delta_\alpha) = 0. \\ \Sigma\,(P + \Delta_P)\sin.\,(\alpha + \Delta_\alpha) + \Sigma\,(R + \Delta_R)\sin.\,(\alpha + \Delta_\alpha) + \\ \Sigma\,(S + \Delta_S)\sin.\,(\alpha + \Delta_\alpha) = 0. \end{aligned}\right\} \ldots\ldots(B)$$

Each of these terms, as $(S + \Delta_S)\cos.\,(\alpha + \Delta_\alpha)$ may be expanded as $(S + \Delta_S)\cos.\,(\alpha + \Delta_\alpha) = S\cos.\,\alpha\cos.\,\Delta_\alpha - S\sin.\,\alpha\sin.\,\Delta_\alpha + \Delta_S\cos.\,\alpha\cos.\,\Delta_\alpha - \Delta_S\sin.\,\alpha\sin.\,\Delta_\alpha$.

Now suppose the Δ all relatively small, then we could write $(S + \Delta_S)\cos.\,(\alpha + \Delta_\alpha) = S\cos.\,\alpha - \Delta_\alpha S\sin.\,\alpha + \Delta_S\cos.\,\alpha$, and similarly with the other terms of (B), and taking Equations (A) into consideration we thus obtain from Equations (B) the difference equations for each joint:

$$\left.\begin{aligned} \Sigma\,\Delta_P\cos.\,\alpha + \Sigma\,\Delta_R\cos.\,\alpha + \Sigma\,\Delta_S\cos.\,\alpha - \Sigma\,\Delta_\alpha P\sin.\,\alpha \\ - \Sigma\,\Delta_\alpha R\sin.\,\alpha - \Sigma\,\Delta_\alpha S\sin.\,\alpha = 0 \\ \Sigma\,\Delta_P\sin.\,\alpha + \Sigma\,\Delta_R\sin.\,\alpha + \Sigma\,\Delta_S\sin.\,\alpha + \Sigma\,\Delta_\alpha P\cos.\,\alpha \\ + \Sigma\,\Delta_\alpha R\cos.\,\alpha + \Sigma\,\Delta_\alpha S\cos.\,\alpha = 0 \end{aligned}\right\} \ldots(C)$$

If the framework be determinate, then, since the P and α are known, the R and S determinable from the Equations (A) or their equivalents, the Δ_α (except for the reactions) very approximately determinable from the elastic displacements of the joints, for the given materials and sections, due to the stresses S, temperature and other causes, and the Δ_P either zero or definitely related to these displacements, Equations (C) suffice to determine our remaining unknowns, that is to say, the quantities Δ_R, Δ_{α_R}, and Δ_S.

The above result suffices for the solution of the problem, but it is capable of an exceedingly simple and objective interpretation which extends at once to the calculation of these changes in stress all the methods and resources, graphical and analytical, already developed for the calculation of the ordinary stresses. Consider the following:

If we inspect Equations (C) and compare them with Equations (A) we perceive that they are simply the equations of equilibrium at the joints of the given framework subject to the loads formed by taking the changes in the original loads, together with the further loads $\Delta_\alpha S$ at right angles to the corresponding bars. The resulting stresses in the framework under these loads are the desired changes in stress. So we have only to find this differential loading, which is readily done when the displacements of the joints have been determined (as by a Williot

diagram), and then we may proceed with the solution of the stresses by any of the usual methods, graphical or analytical, obtaining thus the change in stress resulting from change in form of the framework, in the case of any determinate framework. Mr. Cilley.

Applying the foregoing to the three-hinged arch of the second illustration of the paper, to determine the changes in stress due to change in figure resulting from a fall in temperature, we have the following: For a fall in temperature of 100° Fahr., if the arch did not alter its form, we would have a reduction of span of 0.4×0.065 ft. (see page 405) and a consequent descent of the middle hinge of $\frac{12}{40}$ of this, or 0.12×0.065 ft. But we suppose the span actually remains constant, and therefore the middle hinge must sink further, $0.2 \times 0.065 \times \frac{20}{12} = 0.333 \times 0.065$ ft. The total sinking of the middle joint will therefore be $(0.333 + 0.120) \times 0.065$ ft. $= 0.030$ ft., nearly. That is, all bars of the left half will rotate through the common angle,

$$\text{tan.}^{-1} \frac{0.030 \times 20}{20^2 + 12^2} = \text{tan.}^{-1} \frac{0.6}{544} = \text{tan.}^{-1}\, 0.001103$$

(about $3\frac{3}{4}$ minutes), in a right-handed direction, and all bars of the right half will rotate through the same angle in a left-handed direction.

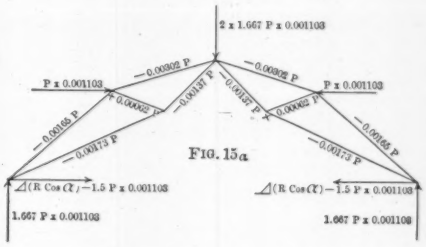

FIG. 15a

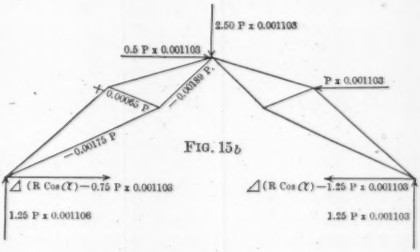

FIG. 15b

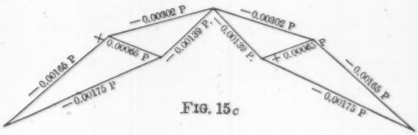

FIG. 15c

Consider, first, the consequent changes in the stresses under a full loading of P on each joint (loading L, C, R).

We have in Fig. 15*a* the differential loading for this case and the resulting stresses which are the changes in stress sought. The value

Mr. Cilley. of $\Delta\,(R \cos.\ \alpha)$, or the change in the horizontal thrust, not given in Fig. 15*a*, is 0.00444 *P*, which permits of ready verification. Next consider the loading *C R* (*P* on center and right joints) under which bars *b*, *c* and *e* (see Fig. 2) have their greatest stresses.

We have, in Fig. 15*b*, the corresponding differential loading and the resulting stresses in the bars *b*, *c* and *e*, which are the changes in stress in these bars sought. The change in the horizontal component of the reaction in this case is found to be $\Delta\,(R \cos.\ \alpha) = 0.003125\ P$, which may easily be verified otherwise.

Putting down the changes in stress in the bars *a* and *d* under the loading *L C R*, and in *b*, *c* and *e* under the loading *C R* (and *L C*) we have, in Fig. 15*c*, the additions to the maximum stresses to be made on account of a change in temperature of 100° Fahr. Compare this with Fig. 15. Taking a change in temperature of only 50° Fahr. (that is, a range of temperature of 100° Fahr.), as in the case of the two-hinged arch, we find our additions are (compare Fig. 10) 0.09% in bar *e*, as against 22% for the two-hinged arch; 0.09% in bar *c* as against 20%; 0.09% in bar *b* as against 10%; 0.09% in bar *a* as against 11%; 0.035% in bar *d* as against 6%, to say nothing of no stress here to compare with the 45% temperature stress in bar *z* of the two-hinged arch.

This comparison, showing the stresses in the case of the three-hinged arch resulting from temperature changes to be only about one-two hundredth of the ordinary temperature stresses of the two-hinged arch, will be found a sufficient answer to Mr. Lindenthal's statement:

"If the author will trace the strains, due to the falling and rising of the center hinge from changes of temperature, he will find them by no means of negligible smallness, even for the large ratio of rise to span in his inconclusive figure."

But this is not all. The "temperature" (?) stresses we have just traced out in the three-hinged arch also occur in the two-hinged arch, in addition to those already calculated. Let us now develop the theory of these changes.

The theory of the changes of stress in indeterminate frameworks due to changes in form is not quite so simple as that for the determinate frameworks just set forth, but is based on the latter. At a first glance it would appear that one might simply find the differential loading for the indeterminate framework, and thence obtain the changes in stress sought simply as the stresses in the given indeterminate framework under this differential loading, on the supposition of no stress under no loading. But this is not quite correct as the following analysis will show.

We have seen (page 366), that the difference of the figure length L_i from the actual length l_i of a superfluous bar is expressed by the equation

$$\sum_{f=1}^{f=m} \frac{S_f'^{(i)}\, S_f\, l_f}{E_f\, A_f} = L_i - l_i \qquad \ldots\ldots\ldots\ldots\ldots\ldots (D)$$

provided only negligible changes in direction of the bars occur. If, as a result of the actual changes in direction of the bars, the stresses actually become $S_f + \Delta_{S_f}$ and the coefficients $S_f'^{(i)}$ become $S_f'^{(i)} + \Delta_{S'_f(i)}$ then we must have Mr. Cilley.

$$\sum_{f=1}^{f=m} \frac{(S_f'^{(i)} + \tfrac{1}{2}\,\Delta_{S'_f(i)})\,(S_f + \Delta_{S_f})\,l_f}{E_f A_f} = L_i - l_i \ldots\ldots\ldots (E)$$

And supposing, as before, that the Δ are small, we readily derive the simple difference equations

$$\sum_{f=1}^{f=m} \frac{S_f'^{(i)}\,\Delta_{S_f}\,l_f}{E_f A_f} + \frac{1}{2}\sum_{f=1}^{f=m} \frac{\Delta_{S'_f(i)}\,S_f\,l_f}{E_f A_f} = 0 \ldots\ldots\ldots\ldots (F)$$

These Equations (F) together with Equation (C) actually determine the changes in stress, Δ_{S_f}. The second term on the left-hand side of Equations (F), which would have to be zero if the Δ_{S_f} were simply the stresses for the differential loading on the supposition of no stress under no load, shows the error that supposition would involve.

Instead of the purely analytic procedure of combining Equations (F) and (C), the following procedure is likely to prove preferable.

The stresses in tne non-superfluous bars are expressed in terms of those in the superfluous bars by the equations (see p. 371).

$$S_f = S_{f_o} + \sum_{i=h}^{i=m} S_i\,S_f'^{(i)} \ldots\ldots\ldots\ldots\ldots\ldots (G)$$

on the supposition of no change in form. Actually, we must have

$$S_f + \Delta_{S_f} = S_{f_o} + \Delta_{S_{f_o}} + \sum_{i=h}^{i=m} (S_i + \Delta_S)\,(S'_f{}^{(i)} + \Delta_{S'f(i)}) \ldots (H)$$

and again, supposing the Δ small, these equations give the difference equations

$$\Delta_{S_f} = \Delta_{S_{f_o}} + \sum_{i=h}^{i=m} \Delta_{S_i}\,S_f'^{(i)} + \sum_{i=h}^{i=m} S_i\,\Delta_{S'_f(i)} \ldots\ldots\ldots (K)$$

Now these Equations (K), together with Equations (F), may form the basis of our solution. In them the Δ_{S_f} are the quantities sought. The $\Delta_{S_{f_o}}$ being the changes in stress of the framework composed of the non-superfluous bars, under the given loading, due to the change of its form for the joint displacements of the given indeterminate framework under that loading; and the $\Delta_{S_f(i)}$ being the corresponding changes in the coefficients $S'_f{}^{(i)}$, they are both determinable by the method for determinate frameworks, explained previously.

As to the solution of Equations (F) and (K), besides the analytic methods, we may readily apply to them any of the usual graphic methods for the determination of stresses in indeterminate frameworks. The similarity of their forms to those of the equations for

Mr. Cilley. stresses in indeterminate frameworks will suggest the manner of utilizing the same methods. Thus one may take advantage here, as in the case of the calculation of changes in stress accompanying changes of form in determinate frameworks, of all the accumulated methods and resources, graphical and analytical, now in our hands for the calculation of the ordinary stresses on the supposition of no change in form.

The writer regrets that his present limited time prevents his illustrating the use of these methods by a calculation of the additional stresses in the two-hinged arch of the paper, resulting from the flattening of that arch accompanying a fall of temperature. It is evident, however, that they would be quantities of the same proportions as those already found for the three-hinged arch.

In concluding this brief exposition of the theory of the calculation of changes in stress in frameworks accompanying small changes in form, the writer believes he can commend its use for cases where the actual changes are really quite considerable, that is to say, for the approximate calculation of the corrections to be given to the stresses in flexible frameworks, as slender arches and stiffened suspension bridges. Although the work is laborious, it is only laborious in the same way and degree as the work of the usual methods for the ordinary calculations on which it is based, and it certainly is far simpler than the application of higher mathematics called for by the present rudiments of methods, proposed or used in such cases. The somewhat sketchy character of the writer's present exposition he trusts will be pardoned as the treatment of the subject here given is wholly new, so far as the writer knows, and was developed, at very brief notice, by him, for this discussion.

Before leaving the subject of temperature stresses the writer desires to note his exception to Mr. Lindenthal's quantitative statement regarding the extra material required to meet the three-hinged arch temperature stresses in the cases of the Washington arch and of the Niagara Falls road bridge (or Clifton) arch. Such claims are too easily made to be acceptable when unaccompanied by any figures in their support, and, above all, when rendered doubtful by general analysis, as in the present case.

To Mr. Lindenthal's claim, that "a proper comparison" of the strain sheets "of the two types of arch treated by Mr. Cilley" (two and three-hinged), demands a correction "for the large bending moments resulting from the immobility of the hinges, incorrectly ignored by him," the writer feels that he must strongly take exception. Mr. Lindenthal forgets that in the paper it was particularly specified that only ideal frameworks with frictionless joints were under consideration. Moreover, the pin is not a necessary connection member for this purpose. Thin plates have not only been proposed, but also used.

In the Kaisersteg bridge* at Oberschoenweide, near Berlin, designed by Müller-Breslau, such a plate hinge was used, and in France an entire set of joints has been constructed recently with such plates, in a bridge of 132-ft. span, described in the *Annales des Ponts et Chaussées.* Mr. Cilley.

Thus, while a correction is necessary where pins are used, they are really unnecessary indeterminate adjuncts, and therefore are not properly to be considered in such a comparison as that made in the paper.

Mr. Lindenthal says that temperature stresses and hinge friction require additions to the sections of both two and three-hinged arches, which cannot be ignored, and follows this with the somewhat extreme statement—"When made (these additions), the two-hinged arch will in every case show great economy over the three-hinged arch." Again, the writer feels that we are entitled to the figures in proof of this bare assertion. The analytic considerations, thus far exposed, by no means support it.

Mr. Lindenthal holds that it "is not necessary, either for safety or economy," that the strains in an indeterminate framework "be determined with the same exactness as in statically determined structures." If Mr. Lindenthal thus unfairly favors indeterminate designs, it may well be that he will find in them the higher economy he claims, but it is certainly competent for the opponents of such designs to take exceptions to such procedure. It must be remembered that it does not take much favoring to eat up the small margin of a few per cent. which will ordinarily limit the difference in material required in the two cases. The smallness of this difference is precisely the reason why the writer argues rather the lack of economic advantage in the use of indeterminate forms, than its presence in the use of the determinate forms.

Mr. Lindenthal states that "no failures of bridges, caused by their indeterminateness, have yet occurred." Has he not, perhaps, overlooked some cases? Partial failures due to indetermination have been anything but wanting. Many a weak member has been broken or has broken its connection when carried along by the movement of some heavy member to which it was improperly attached; many a case of initial strain, permitted by the use of redundant members, has resulted in rupture; many a continuous floor system has failed in the rivets, under the stresses actually coming upon it, for which it was not designed; and in accidents like the buckling of the lower chords of the stiffening trusses of the Brooklyn bridge over the towers, we have illustrations of the consequences of indetermination in the case of more important parts. It may be said that these were due, not to the use of indeterminate construction, but to faulty calculation, or neglect to calculate at all, or to faulty adjustment. But these things arose purely from the indetermination and may therefore properly be

* Described in *The Engineering Record* of Feb. 17, 1900.

Mr. Cilley. charged against it. Far from indetermination never being a cause of failure, it has been a fruitful cause, especially in other structures than bridges, where temperature strains have worked havoc.

We now come to the last of Mr. Lindenthal's objections, which is to deflection as a criterion of rigidity. He says: "What is a rigid bridge? If we take rigidity in metallic bridges to mean absence of vibration, then deflections are a deceptive criterion."

If this were merely a question of difference of definition of the word "rigidity," it would have no further importance, but Mr. Lindenthal continues: "The public considers a bridge which vibrates, as an inferior structure; and one which does not, as a superior one. This ought to be the guide, also, to the engineer." But this argument of "*Vox populi vox Dei*" is hardly convincing, and Mr. Lindenthal's illustrations only serve to increase the writer's distrust.

Vibration, far from being an indication of anything defective, is the evidence of a highly elastic and rigid constitution. Only rigid and elastic large bodies vibrate rapidly, and therefore, in a fashion very noticeable to us. The vibrations of a slack rope are not marked, but it is scarcely rigid. A Peruvian rope suspension bridge is not notable for its vibration, but even the public would scarcely regard it as superior. Many a loose-jointed wooden structure is free from vibration, while in similar metal structures it is most marked, but the wooden structure is hardly therefore the superior one.

Wherever great rigidity, elasticity and lightness go together, vibration is likely to be noticeable. Where, on the contrary, there is loose or slack construction, imperfect elasticity and great mass, vibration will be imperceptible. The writer can scarcely believe that Mr. Lindenthal had seriously considered the bases of vibration before he thus proposed it as the criterion for distinguishing bad from good construction. Large deflections, which mean flexible construction, really are objectionable, because they interfere seriously with high speeds, and, moreover, allow of dangerous swaying in high winds. But rapid vibration is an evil affecting the imagination far more than it affects the structures. Moreover, Mr. Lindenthal's illustrations show that marked vibration occurs equally in indeterminate and determinate structures. It is, in fact, a matter largely independent of the presence or absence of superfluous bars.

In concluding this part of his reply, the writer feels justified in protesting at the statements wholly unsupported by evidence, and some of which analysis has shown to be incorrect, which characterize this part of the discussion. It was his hope that those who made counter claims to the arguments in the paper would adduce their evidence in such form as to permit of its independent consideration. In the absence of such evidence it can scarcely be expected that these

claims will receive much consideration, and, certainly, we cannot be expected to accept them. Mr. Cilley.

In replying to Professor Ritter's comments, the writer desires to express at the outset his appreciation of the very fair and reasonable position taken by him, as well as of the entire absence of any misapprehension or misconstruction of the paper in these comments. In the hands of men as able and as honest as Professor Ritter, the use of indeterminate frameworks loses many of its objections. But, since few engineers favoring indeterminate construction are equally well informed and fewer are as fair, its use is generally subject to the full force of the writer's objections.

Some of the points touched on by Professor Ritter have been considered by the writer in his reply to Mr. Lindenthal, but there remain several points which it is desirable to consider here.

The shorter unsupported lengths of the struts is an advantage which has frequently been cited in connection with lattice webbing, and as a justification of the European custom of riveting members together wherever they may cross. But it seems to be questionable whether the apparent advantage thus obtained is not illusory. As was long ago pointed out by Maurice Levy, the use of multiple systems of webbing results in a reduction of the sections of the posts which may well much more than offset the reduction of free length due to attachment at points of intersection. And this is apart from the fact that the free lengths for side deflections (deflections out of the plane of the truss) are not reduced through this arrangement in anything like the degree that the free lengths for deflections in the truss planes are reduced. But this is not all. Such attachment of points of intersection results in bending moments (when it does not result in anything more), which are rarely if ever considered, although sometimes very large. When these are taken into consideration it will be found that the consequent added secondary stresses much more than offset any advantage from shortened free length. Moreover, this practice of riveting every member to every other member wherever there is a chance becomes a wholly indiscriminate and blind action which sometimes entails serious unforeseen results, and has been responsible for no small part of the failures in indeterminate construction. The fact that not more than two passing co-planar members can be connected at the same point without entailing, in addition to bending stresses, direct axial stresses, is usually overlooked or not appreciated.

The writer's view is, that as far as possible we should design only as we can and do calculate and understand, and that the procedure he is now criticising, the consequences of which are neither calculated nor understood, should be put aside by all who prefer the guidance of their intelligence to slavish copying of ill-founded methods of the past. Some day it will be recognized that every unnecessary connec-

Mr. Cilley. tion is only an added restraint upon the free exercise of function, and that, as the writer has elsewhere pointed out, "Statical indetermination in a structure is always to be regarded as self-interference with efficiency."

In regard to the objections to the expense of hinges, constructed as at present, with pins, the writer ventures to suggest that this difficulty, as well as that of the shocks at the hinges, mentioned by Professor Ritter, may be overcome by the substitution of thin plates for pins. Flexible joints by no means depend on the use of pins for their realization, as is so commonly assumed.

Regarding discrepancies of length of members, the writer would point out the consequences in this direction which the theory of probability tells us we must expect. We know that an error ε in the length of any member of a statically determined framework, will change the distance, between any two of its joints not directly connected, by the amount $S'\varepsilon$ (where S' is the stress in the member due to a pair of equal and opposite forces of unity applied each at one of the two joints). Now suppose each bar f has a corresponding probable error ε_f in its length (which error may equally well be + or —), then the probable resultant error Δ, in the distance between the two joints, as a consequence of all these probable single errors ε_f, we know from the theory of probability will be

$$\Delta = \sqrt{\Sigma\,(S'\varepsilon)^2}$$

Let us apply this to an example:

Suppose we have a two-hinged arch of, say, 500 ft. span, and 90 ft. rise, with parallel chords 15 ft. apart, and that it is built out from the abutments to meet at the center. Suppose that we have forty panels and that the probable error of each member is only $\frac{1}{64}$ in. This is moderate for even the best of work, that is to say, where all lengths are determined by milled surfaces brought firmly into contact for riveted joints, or by carefully bored pin bearings. Very careful, repeated measurements made by the writer with an excellent steel tape, firmly clamped at one end and subject to a uniform pull by means of a spring balance at the other end, have convinced him that errors in measurement alone are likely to reach this amount. For simple riveted connections the work would be sensibly less accurate. Here it will readily be seen that the average value of S' for the chord members is about two-thirds, while that for the web members is less than one-fifteenth, but more than one-twentieth. There will be 80 chord members, which alone would cause a probable error in the closing middle chord bar of

$$\frac{2}{3}\sqrt{80} \times \frac{1}{64} = \frac{3}{32} \text{ in., about.}$$

With a simple triangular system, as we will suppose, there would also be about 80 web members, and taking for them the smaller average

value of S' as one-twentieth, the probable error in closing of the middle chord bar due to their errors would be Mr. Cilley.

$$\frac{1}{20}\sqrt{80}\times\frac{1}{64}=\frac{1}{128}\text{ in., about.}$$

The resultant probable error for both chords and webbing would be

$$\sqrt{\left(\frac{3}{32}\right)^2+\left(\frac{1}{128}\right)^2}$$

(since chord and web members are equally numerous) or practically $\frac{3}{32}$ in., as for the chords above. This gives an idea of what may be expected in the final result from a given degree of accuracy of workmanship, and so tells us what degree of accuracy it is necessary to maintain in a given case. Some such method should be used where indeterminate frameworks are constructed.

Professor Ritter states that "In general, structures with hinges (arches or cantilevers) are less stiff than those without hinges." The writer feels that this statement, reasonable as it appears at first glance, cannot, after all, be so readily accepted. It involves, first, the question of what are comparable structures; and, second, the question of what basis of judging stiffness is to be used. Confining ourselves to deflections for this latter, as the writer has done, there still remains the question of where the deflections are to be taken. It does not suffice, particularly with arches, to compare only center deflections, for the deflections of non-central points due to non-symmetric loadings are likely to be larger than those at the center, as in the writer's illustration. Such being the case, how shall we combine these? Shall we take the average of deflections at a series of given intervals, or shall we simply compare the greatest deflections wherever occurring? The writer would again refer to his remarks on page 374, in this connection.

In concluding this reply to Professor Ritter, the writer would suggest that perhaps "the extensive adherence of American engineers to stiff joints in upper chords, as well as rigid attachment of floor-beams, stringers and lateral, portal and sway bracing," is largely due to practical (shop) considerations, together with recognized objections to the use of pins in many cases, especially where adjustable members would be introduced. The abuse of adjustable members has proved to be a serious matter in this country. The writer has been informed of a case where a section of the bottom chord was relieved of its part of the stress and made slack by the over-tightening of counters. And, as is well known, small counters and their connections are often seriously overstrained in this manner.

Perhaps, when a practical system of making connections with thin plates has been developed, a matter which will require much time and experience, even with the best of good will on the part of designers,

Mr. Cilley. engineers will sufficiently appreciate its merits in eliminating secondary stresses, to give it preference over the present system of deceptive connections, whose strength is always much less than it is supposed to be, and whose additions to stiffness are only obtained at the expense of notable and unconsidered increases of stress in the main sections.

The comments of Professor Dietz are directed principally to pointing out what numerous factors enter into the final determination of the cost of a structure. To these the writer at once assents, but he discovers in them little or nothing affecting the validity of the arguments he has advanced in the paper. Most of the factors mentioned by Professor Dietz are equally factors with both determinate and indeterminate frameworks, and, therefore, do not influence their comparison, provided only that in any detailed comparison of two designs they are actually made equally factors in the two cases. As to the fact that the superstructure is only an element of the total, as far as it is of moment at all, it is a factor wholly favorable to determinate construction. Nothing like as carefully built and expensive foundations and supports are required for the determinate as for the indeterminate structures. The writer knows of no practical factor except the large (?) expense of pins which can be cited as adverse to determinate construction, and, aside from the fact previously noted, that their use is not necessary, the saving in cost of erection resulting from it may exceed the cost of manufacture.

Regarding Professor Dietz's detailed comparison of a two and three-hinged arch, the writer begs to note that the placing of the middle hinge below the top chords has very probably deprived the three-hinged arch of a very legitimate advantage it might have had. Moreover, until we know how the sections were determined we cannot know whether the comparison is really fair in this respect. These considerations affect the stated deflection also, but it is in a further respect misleading. It is simply the middle deflection and, as previously noted, very possibly, if not probably, is not the greatest deflection.

Finally, the writer is not inclined to agree at all with Professor Dietz's apparent view that only statistical comparisons of numerous complete projects can be conclusive. There is not now and there probably will not be (in the writer's opinion) within the next century, enough fairly comparable data to enable a definite conclusion to be drawn therefrom respecting the relative merits of determinate and indeterminate frameworks in general. Worked-out cases are almost invariably not fairly comparable. The writer holds that the best, the most reliable, if not the only fair basis of comparison of the relative merits of determinate and indeterminate frameworks must lie in such comparison of ideal structures as he has made. Actual or complete designs are subject to so many irrelevant variations that their comparison necessarily carries with it no general conclusion.

Mr. Cilley.

The writer desires to express his appreciation of the brief but very correct summing up of the paper included in the comments of Prof. Joseph Sohn. It is a great help to grasping the main points of such a paper, necessarily somewhat long and complex in its complete form, to have its essential points brought out in so concise and clear a restatement as in the present instance.

The writer is pleased to learn of Professor Jung's adherence to the use of determinate rather than indeterminate systems. Italy has long been a leader in the intelligent application of science to the arts, and notable for her freedom from the old dogmatic beliefs which in many other countries still so largely maintain their hold. The writer well remembers that many of the most lucid and valuable of the modern additions, both to the theory of structures and to the general theory of elasticity, have come from Italy, and believes that in the extension of the use of determinate construction and of intelligent designing Italy will again be in the front.

Vol. XLIII. JUNE. 1900.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

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No. 874.

EXPERIMENTS ON THE PROTECTION OF STEEL AND ALUMINUM EXPOSED TO WATER.

By A. H. SABIN, Assoc. M. Am. Soc. C. E.

PRESENTED DECEMBER 20TH, 1899.

WITH DISCUSSION.

This paper states the results of the continuation of a series of experiments to determine the comparative durability of paints and varnishes, which experiments were begun in February, 1895, and were made the subject of a paper read before this Society on November 4th, 1896.* The plates described in that paper had been exposed to the action of sea water in the New York Navy Yard for a period of six months; those to be described now were chiefly triplicate sets, one set of which was exposed under the surface of the sea water in the New York Navy Yard, another was similarly placed in the Norfolk Navy Yard, while the third was placed in fresh water, in Lake Cochituate, near Boston, Mass. All were placed in the water during the last half of June, 1897. Those at Norfolk and Boston remained untouched until July, 1899, a little more than two years; but those in the New York waters were in cages suspended to a float which was accidentally sunk in July, 1898, and more than half the plates were lost; the remainder

* *Transactions*, Am. Soc. C. E., Vol. xxxvi, p. 483.

were taken out July 21st, 1899, after an immersion of exactly thirteen months.

The consent of the Navy Department was obtained for placing the plates in the Navy Yards, and in this whole matter the writer is under great obligations to Naval Constructor F. T. Bowles and Assistant Naval Constructor R. M. Watt, of the New York Yard, and to Naval Constructor A. W. Stahl, of the Norfolk Yard. The Boston set was placed by the courtesy of Desmond FitzGerald, President, Am. Soc. C. E., Engineer of the Sudbury Department, Metropolitan Water Board of Massachusetts.

The assistance rendered by these gentlemen was essential to carrying on the experiments. The frames or cages, containing the plates of steel and aluminum which had been painted, were placed in and removed from the water by their assistants; the New York plates, on removal from the water, were inspected by Mr. Bowles and Mr. Watt, also by representatives of some of the technical journals, and by several engineers; the Norfolk set was examined by Mr. Stahl and by an expert, under his directions. The Engineering Department of the Metropolitan Water Board had a representative on the ground to inspect the plates at Lake Cochituate, but, owing to unexpected difficulties encountered in getting the plates from the water, it was found inconvenient to make a local inspection.

With the exception of the ships'-bottom paints, which were applied in the New York Navy Yard, the work of preparing and applying the coatings was done by Edward Smith & Company, of New York City, who paid all the expenses incident to the whole work. The aluminum plates were furnished free of charge by the Pittsburg Reduction Company.

The plates were suspended in open cages or racks, and were in a horizontal position one above another, about 2 ins. apart. They fitted tightly in the racks, and each plate was supported at the four corners. The racks were suspended about 6 ft. below the surface of the water, but at Lake Cochituate they were laid on the bottom, which was hard, about 20 ft. below the surface, and the plates were vertical. Besides the plates lost at New York, one rack containing 25 plates was lost at Norfolk. It was suspended by two $\frac{3}{8}$-in. galvanized iron chains, which rusted off. This gives some idea of the severity of the exposure. All the Boston plates were recovered.

In the salt-water tests it was found that barnacles and other organisms attached themselves to the lower sides of the plates. In the Norfolk Navy Yard the action of these was so severe as to destroy the paint on the under sides of all the plates, with the exception of those coated with the Sabin Pipe Coating, which did not seem to be affected, although oysters 4½ ins. in length were found growing on it. When these were removed the coating was found to be intact. But with this exception, it should be remembered, in looking over Table No. 1, that only one side of the plates in the Norfolk set is described, the coatings on the other side being uniformly destroyed, while in the New York and Boston sets both sides of the plates are included in the description.

Such varnishes as are mentioned in this paper are made by melting, at a temperature of 700° to 900° Fahr., certain resinous substances, such as Kauri, Manila and Zanzibar resins, and the asphaltic mineral known as gilsonite; 100 lbs. of resin is usually melted at once, and to this is then added a quantity of linseed oil, also hot, the amount of which varies according to the kind of varnish to be made. If 20 galls. of oil is used the product is called a 20-gall. varnish; if 30 galls., it is a 30-gall. varnish; and so on. The mixture of resin and oil is heated for some hours until combination occurs, and the product is then thinned with spirit of turpentine, which is regarded as a vehicle. Varnishes made with 20 galls. of oil, and with Manila, Kauri and Zanzibar resins, would be designated as 20 M., 20 K., and 20 Z., respectively; and so on. Enamel paints are made by grinding pigments in these varnishes, exactly as oil paints are made by grinding the same pigments in linseed oil.

Inasmuch as it has been believed that the process of baking adds to the durability of these coatings, in a few cases duplicates were prepared, one of which was baked and the other allowed to dry at the ordinary temperature.

The general scheme was to apply to a set of four plates a set of three varnishes, 20 K., 30 K. and 40 K., and pure raw linseed oil. Then, for another similar set, these same liquids were mixed, by grinding with white zinc; another set was prepared with white lead; another set with ultramarine blue; another set with graphite; and so on. This ought to show whether one pigment is better than another, and which vehicle is the best. Besides these, plates were painted with

pure red lead in pure linseed oil, with two mixtures of red lead and white zinc, with purple oxide of iron (crocus) in oil, and with Prince's metallic oxide of iron, which is a very well-known pigment consisting of iron oxide mixed with various silicates, in oil.

Besides these coatings of known composition, two popular and widely-known proprietary paints, the Eureka paint and the graphite paint made by the Detroit Graphite Manufacturing Company, were tried. These oil paints and proprietary paints were presumed to afford a sort of standard by which the other coatings could be judged.

The coating material described in the table as "Spar," is one of the well-known class of spar varnishes used for exterior work, and the kind used was made by Edward Smith & Company. The "I. X. L. No. 2" is a well-known interior varnish. The substance indicated by the initials "D. M. C." is Edward Smith & Company's "Durable Metal Coating," which is a varnish in which a considerable amount of gilsonite has been substituted for a corresponding quantity of resin. "Parahydric" is a coating similar to "Durable Metal Coating," but containing less oil, and has been used in painting the interior of water-pipes. "S. P. C." is the Sabin Pipe Coating, a varnish enamel which is baked on at about 400° Fahr., and which has been used on some of the largest hydraulic works in this country and is largely used in coating pipes on United States naval vessels. "Keystone" is a well-known pigment, probably ground slate, and was used to furnish a pigment composed of silicates for comparison. The iron oxide used is the purest commercial sesquioxide, containing over 95% oxide of iron. The purple oxide of iron is oxide which has been subjected to prolonged heating, and is supposed to be anhydrous. The "iron oxide in shellac" mixture was prepared from a formula furnished by Naval Constructor Bowles; the shellac is pure "D. C. Shellac" in grain alcohol. The paints known as Rahtjen's, McInnes' and Holtzapfel, are anti-corrosive and anti-fouling ships'-bottom paints, and were furnished and applied by the New York Navy Yard.

The nomenclature and abbreviations of Table No. 1 are as follows:

20 K.	= 20-gall.	Kauri	Varnish.
30 K.	= 30 "	"	"
40 K.	= 40 "	"	"
20 M.	= 20 "	Manila.	"

30 M.	= 30-gall. Manila Varnish.
40 M.	= 40 " " "
20 Z.	= 20 " Zanzibar "
30 Z.	= 30 " " "
40 Z.	= 40 " " "
D. M. C.	= Durable Metal Coating.
S. P. C.	= Sabin Pipe Coating.
Um. Blue.	= Ultramarine Blue.
W. Z.	= White Zinc.
W. L.	= White Lead.
A.	= Pure Aluminum.
A. A.	= Aluminum Alloy, 95 per cent.

The plates used for the purpose were of uniform size, 12 x 20 ins.; part of them were aluminum and the remainder steel, about $\frac{1}{8}$ in. in thickness. Since it is well known that aluminum is not acted on by fresh water, the aluminum plates were all put in the New York and Norfolk sets. The plates numbered from 151 to 163, inclusive, in the New York set, correspond to those numbered from 176 to 187, inclusive, in the Norfolk set, and are pure aluminum; those numbered from 164 to 175, New York, correspond to 188 to 200, Norfolk, and are aluminum plates alloyed with 5% of other metal.

Besides these regular sets of plates, a cage containing 24 plates, part steel and part aluminum, which had in 1896 been exposed for six months in the New York Yard, and are described in the writer's paper in November, 1896, were again exposed with the New York set. Half of these were lost by the accident already referred to, but the remainder are described in Table No. 1. Of these, Nos. 104, 105, 113, 122 and 124 are aluminum,* and the rest are steel. All are distinguished in the table by the date (1896) after the number.

As two entire cages of steel plates were lost in the New York Yard and one in the Norfolk Yard, the descriptions of the aluminum and the "1896" plates have been put in the otherwise vacant spaces of the table. All the plates, except those coated with Sabin Pipe Coating, which had two coats, received three full coats, well dried between coats. The red-lead paint used weighed about 35 lbs. to the gallon, and was put on with the plate in a horizontal position, on the top side of the plate. After the paint had set, the plate was turned over and

* See paper already referred to.

the other side was painted. The steel plates were all of the grade known as pickled and cold-rolled, and were cleaned with a steel wire brush. They were not pickled after they had left the steel mill, but their surfaces were perfectly clean and bright after being brushed.

Results.

In most cases, the protective coatings seem to begin to deteriorate on the surface, which becomes rough; then little blisters appear, which are caused by the separation of the last coat from those beneath; finally the undercoat blisters, in which case it is found, almost invariably, that rust has formed under the blister. If, however, the coating is porous—and this seems to be the case with the ordinary oil paints—the water reaches the metal and causes rust. This throws off the paint film, and the corrosion spreads rapidly in this way.

Protective coatings, such as are applied with a brush, are of three classes:

1. Pigment paints, in which the vehicle or liquid part is either linseed oil (or an inferior substitute) or varnish, and with this liquid portion is mixed mechanically a powdered solid, usually a mineral substance ground to a fine powder;

2. Red lead, which is a mixture of red lead and linseed oil, and which does not dry by oxidation as do other oil paints, but sets like a cement; and

3. Varnishes.

The linseed oil in ordinary oil paints dries by the absorption of oxygen to a tough leathery substance, but the film thus formed is soft and gummy, and is quite porous. Pigment is added to the oil for three reasons. It makes the liquid thicker and more viscous, thus enabling the operator to put on a thicker film; it makes a harder film, which will better resist abrasion; and the particles of pigment tend to fill up the pores of the film, thus rendering it more continuous.

Varnish forms a much more continuous (less porous) film than oil, hence it is better adapted for use in paint, and this is shown conspicuously by an examination of these tests, where the oil paints have failed, without exception, while the corresponding varnish paints are, in most cases, in good condition. The character of the pigment does not seem to have much influence. All the oil paint samples were so badly rusted that differentiation among them was impossible. It may

be that an earlier inspection would have shown differences, but the present appearance of all these plates is so similar that it seems unlikely, and certainly the varnish paints do not show any great differences in the matter of the pigments, except that white zinc seems to be somewhat the best. The iron oxides, graphites, and pulverized slate are all alike. The red lead, in the Boston and New York sets, is far better than any of the oil paints. The mixtures of red lead and white zinc are markedly inferior to red lead alone. In the Norfolk test, which was much more severe, the red lead had finally been quite destroyed. Deterioration in the case of red lead seems always to start from centers. In the Boston set, the red lead seemed to be in pretty good condition, but would probably not have lasted more than a year or so longer. It shows numerous rust spots. The blisters all start from the metal. But most of the varnish paints are much better than the red lead.

A study of the varnishes applied without pigment seems to show that in the fresh-water exposure the process of baking was, on the whole, of advantage, but not greatly so. In the salt water, the unbaked varnishes were better than the same varnishes baked; this agrees with the results recorded in 1896. The Manila varnishes are distinctly inferior to the Kauri and Zanzibar; the "Durable Metal Coating" is best of all. This is doubtless due, in a large degree, to the fact that this varnish, which is intended especially for the protection of structural steel, is made with a heavy body, and the film is of greater thickness than is the case with varnishes intended for wood work. Its composition has also been very carefully studied and designed to secure great durability, which is of much less importance than other qualities in ordinary varnishes.

By far the best results, however, with the exceptions to be hereafter noted, were obtained from the best of the enamel paints. Here, also, the Manila varnishes are decidedly inferior, and, in the opinion of the writer, these should be excluded hereafter from any such tests, although they make a much better comparative showing on wood.

In the enamel or varnish paints, those made with the more elastic varnishes (those containing the most oil) are decidedly the better. The extreme durability of these is well shown by the "1896" plates. These were first exposed to the air two or three months, then they were in the sea water six months, then exposed to the air nearly a year,

then under water thirteen months, and have since been exposed to the air over sixteen months, making a total of four years, and they are still, to all intents, perfect.

It is true that the air exposures have been indoors, but it was in a building adjacent to a railroad yard, where unprotected iron and even nickel steel is covered with deep rust in two or three months, and the air is often so acrid as to cause coughing. Two years' continuous immersion in fresh water has not injured some of these enamels, and two years in the excessively severe exposure at the Norfolk Navy Yard has left several of them in good condition, a few being practically uninjured.

It is generally true of all the better class of coatings that corrosion begins at the edges of the plate. In the case of aluminum plates, it seems evident to the writer that some of these coatings, notably the "Spar varnish" and the "Durable Metal Coating" have been gradually thrown off by corrosion creeping from the edge, probably from some mechanical injury, under the varnish, a patch of which remains uninjured and apparently without deterioration on the middle of the plate. This fact should not be lost sight of in considering this matter, and is one of the points shown by an inspection of the plates, but not easily brought out in a description.

Undoubtedly, the most obvious and conspicuous, and the most instructive, fact, is the total and absolutely universal failure of linseed oil films, either alone or mixed with any of the numerous pigments which were tried, while the corresponding varnishes and enamel paints made with the same pigments are in fair to good condition. This confirms strikingly the opinion, long held by the writer and many others of greater experience, that varnish films are much more impervious and resistant than any others. The exceptional cases to be noted are:

First.—The "Sabin Pipe Coating," a baked enamel, which is so much superior to the others as to form a class by itself, and

Second.—The extraordinary showing made by pure shellac varnish in the Lake Cochituate test. Shellac varnish is simply a solution of shellac resin, which is chemically an acid substance, in alcohol.

There are many grades of shellac; the one used was what has for many years been known as "D. C." orange shellac, and it was dissolved in pure 97% grain alcohol. Being an acid substance, it is attacked and dissolved readily by the ammonia in the atmosphere. It

is removed easily by soap and water. It has never been considered a durable varnish as ordinarily used on wood work, and it does not stand at all in the sea-water tests, but two years' exposure under 20 ft. of fresh water does not seem to have injured it sensibly. This may be a very serious matter, for, while in this regard it is no better than some other varnishes, such as "Durable Metal Coating," which cost much less money, shellac varnish has some important and exceedingly desirable qualities, which no other varnish has. For example, occasionally, we are confronted with the problem of repainting a section of large water pipe which can be spared from use only a few days. The interior of this pipe is damp. The best that can be done with it is to get out most of the visible water, but the cold surface of the metal will always be damp. No ordinary varnish will stick to such a surface, and corrosion will probably be set up at once; no ordinary varnish, of sufficient durability to be worth putting on, will dry in the limited time at our disposal. Now, shellac is dissolved in a vehicle which has an intense affinity for water, and it is highly probable that a thin film of dew will be instantly absorbed, and will certainly be removed, by the evaporation of the slightly diluted alcohol; and shellac, if applied in a thin coat, dries with the greatest rapidity. Three coats may be applied in eight hours. There is no unpleasant or dangerous odor, though ventilation should be secured, both on account of the risk of fire and because working in an atmosphere of alcoholic vapor produces intoxication. It certainly seems from this test as though we would be justified in using shellac varnish in such a case. It is expensive, of course, and it is very likely that the cheaper grades, which are found in ordinary use to be very much inferior in durability, would not be so efficient. In any case it would not be necessary to use it where the conditions are such that some equally good but slower drying coating can be used.

The metal plates used in these tests were free from rust or scale, and showed in all cases the bright color of the metal. To prepare plates for a paint test, this is the only way which will give comparative results. The writer is positive about this. It is sometimes said that we do not get our metal in this condition when we paint it, and therefore it is proper to make tests on somewhat rusty iron, or on iron with mill scale on it. In the first place, we ought to get the iron in that clean condition, and the writer is not without hope of liv-

ing to see the day when it will be done, in the case of permanent structures; and secondly, he is satisfied, from many years' constant experience in such experiments conducted on a most extensive scale, that it is absolutely impossible to get uniform conditions with rusty metal. A dozen metal plates, even if cut from the same sheet, will not rust uniformly; one will throw off a scale and another will not. The mill scale on a plate will be almost entirely removed from part of a plate by passing it through bending rolls, while it will adhere firmly to another part of the same plate. If this plate had been cut up and used for tests, the paint on one piece would come off and on another would stay on, and the scale is always different on the two sides of a plate.

Exposure tests, such as these, are of much more importance than laboratory tests. The manufacturers of paints and varnishes, some of whom are probably the best experts in this matter, never depend on any but an exposure test. It is by no means impossible that rapid laboratory tests may yet be devised, but such crude ones as have been so far proposed are in most cases of little value. Such a test, for example, is that with caustic alkali. This is a substance unknown in Nature, and no good paint will stand it, while a perfectly worthless paint might be made which would stand it very well. A nitric-acid test is of the same sort. It will simply burn up any organic substance, and some of the best linseed oil paints will yield to it most readily. It would hardly be regarded as a fair test of the comparative health of a dozen human beings to administer to each of them a couple of ounces of nitric acid and keep watch to see which lived longest, yet probably each could take a few drops of it per day without inconvenience. This is about what many of these so-called paint tests amount to. Some laboratory tests are of considerable value, but none is conclusive.

It has frequently been objected to these submarine tests that they are of value only as regards the same conditions, and there is some justice in such a criticism; but it is much weakened by the obvious fact that there is a practical agreement between the fresh-water and the sea-water tests. The latter were most severe, but in most cases the difference has been one of degree only. And in the rather large experience of the writer and his associates, these tests seem to agree in general with aerial exposures, reasonable exception being made in the case of coatings intended expressly for marine or for aerial use.

TABLE No. 1.

Lake Cochituate, Boston.	401–20 K. No rust except where damaged along edges; many very small blisters.	404–20 M. Much rust; coating much injured.	407–20 Z. Not much corrosion, but coating about destroyed.
	402–30 K. Like 401, not quite so good.	405–30 M. Worse than 404; coating nearly destroyed.	408–30 Z. Like 407, but considerably better.
	403–40 K. Like 401.	406–40 M. Not quite so bad as 405.	409–Spar. Like 408, but perhaps a little better.
Navy Yard, Norfolk, Va.	301–20 K. 301 to 310, coatings not destroyed; all considerably injured; blistered in small spots; no considerable corrosion; 301 worst; 306 and 309 best, 307–8 not bad.	304–20 M.	307–20 Z.
	302–30K.	305–30 M.	308–30 Z.
	303–40 K.	306–40 M.	309–Spar.
Navy Yard, New York.	1 (1896)–W. L. in 20 K. Some rust along edges; otherwise in good condition.	16 (1896)–W. Z. in 20 M. One-fifth of one side rusted; all the rest in good condition.	47 (1896)–Spar. Coating firm and good; very little rust.
	2 (1896)–W. L. in 20 K. Like 1.	18 (1896)–W. L. in 8 M. Paint hard and firm; in good condition.	35 (1896)–S. P. C. A little corrosion near the edges; otherwise all right.
		20 (1896)–W. Z. in 30 Z. Good. No blisters; no rust.	113 (1896)–S. P. C. Two small blisters; otherwise good.
		22 (1896)–W. Z. in 20 Z. Good. Some corrosion along edges.	

TABLE No. 1—(*Continued*).

410- I. X. L. No. 2. About like 407.	413-D. M. C. Good, except where broken and injured along edges.	417-Parahydric. Numerous isolated rust spots about ¼ in. diameter; coating otherwise good.	Lake Cochituate, Boston.
411-Shellac. Very excellent condition.	414-D. M. C. Like 413.	418-Parahydric. Like 417.	
412-Raw Oil. Surface generally corroded; many tubercles.	415-D. M. C. Like 413.	419-Parahydric. Like 417.	
	416-D. M. C. Like 413.	420-Parahydric. Like 417.	
310-I. X. L. No. 2.	313-D. M. C. Many small blisters, in outer coat chiefly; very little corrosion.	317-Parahydric. Coating all on; no blisters.	Navy Yard, Norfolk, Va.
311-Shellac. Coating practically gone; badly rusted.	314-D. M. C. Like 313.	318-Parahydric. Like 317.	
312-Raw Oil. Coating destroyed; very badly rusted.	315-D. M. C. Like 313.	319-Parahydric. Like 317.	
	316-D. M. C. Like 313.	320-Parahydric. Like 317.	
124 (1896)-Spar, one side baked. Very few small blisters, otherwise perfectly good.	105 (1896)-Chromium oxide in 20 K., one side baked. A few blisters; otherwise in excellent condition.		Navy Yard, New York.
104 (1896)-W. Z. in 20 K., one side baked. Like 124.			
122 (1896)-W. Z. in 20 K., one side baked. Like 122.			

TABLE NO. 1—(*Continued*)

Lake Cochituate, Boston.	421-S. P. C. Perfect, except where coating is in one or two places broken at edge with corrosion.	425-W. Z. in 20 K. Half the surface, along the edges, blistered, with rust underneath.	428-W. Z. in 20 M. Outer layer of coating nearly destroyed; under coat good.
	422-S. P. C. Like 421.	426-W. Z. in 30 K. Much better than 425; some blisters; little corrosion.	429-W. Z. in 30 M. A few slight rust spots; outer coat blistered.
	423-S. P. C. Like 421.	427-W. Z. in 40 K. Good condition; some blisters in outer layer of coating; no rust.	430-W. Z. in 40 M. About one-fifth rusted; thin rust. Blistered; outer coat chiefly.
	424-S. P. C. Like 421.		
Navy Yard, Norfolk, Va.	321-S. P. C. Perfectly good condition. See note in text.	325-W. Z. in 20 K. Blistered; not very good.	179 A-I. X. L. No. 2. Coating all gone.
	322-S. P. C. Like 321.	176 A-20 K. Three-fourths of coating destroyed; thin rust.	180 A-Spar. Two-thirds of coating gone, but one-third in the middle perfectly good.
	323-S. P. C. Like 321.	177 A-30 K. Like 176.	181 A-D. M. C. One-fifth gone, one-fifth blistered; remainder good.
	324-S. P. C. Like 321.	178 A-40 K. Coating all gone.	182 A-S. P. C. One-tenth gone on one edge; remainder all right.
Navy Yard, New York.			154 A-I. X. L. No. 2. Varnish half gone. Corrosion not deep.
		151 A-20 K. Blistered along edges and a few spots. Varnish firm. Little corrosion.	155 A-Spar. Most of the varnish soft, but some not affected. Not badly corroded.
		152 A-30 K. Much corrosion; some deep. Coating half gone; remainder firm.	157 A-D. M. C. Twenty per cent. blistered around edges. Coating firm; not much corrosion.
		153 A-40 K. Badly corroded; coating nearly all destroyed.	158 A-S. P. C. Excellent. Coating not injured, except by accident in removing from frame.

TABLE No. 1—(*Continued*).

431–W. Z. in 20 Z. Not much rust; outer coat badly blistered; under coat slightly so.	436–W. Z. in 20 K., baked. Almost perfect; still shows glossy surface of varnish.	439–W. Z. in 20 M., baked. Good; coating brittle in places and shows deterioration.	Lake Cochituate, Boston.
432–W. Z. in 30 Z. Better than 431. Outer coat blistered.	437–W. Z. in 30 K., baked. Like 436.	440–W. Z. in 30 M., baked. A little better than 439.	
433–W. Z. in Spar. Like 432.	438–W. Z. in 40 K., baked. Like 436.	441–W. Z. in 40 M., baked. Almost perfect.	
435–W. Z. in Raw Oil. Four-fifths of surface badly rusted; deep corrosion.			
183 A–S. P. C. Perfectly good condition.	187 A–W. Z. in Spar. Like 184.	191 AA–I. X. L. No. 2. Coating all gone.	Navy Yard, Norfolk, Va.
184 A–W. Z. in 30 K. Fine; no rusting nor blistering.	188 AA–20 K. Coating all gone.	192 AA–Spar. Three-quarters gone; like 189.	
185 A–W. Z. in 40 K. Like 184, but discolored.	189 AA–30 K. Three-quarters gone; small patch in the middle all right.	193 AA–Spar. Like 192.	
186 A–W. Z. in 30 Z. Like 184.	190 AA–40 K. Half gone; like 189.	194 AA–D. M. C. One-third badly blistered from edges; remainder good.	
159 A–S. P. C. Like 158 A.	163 A–W. Z. in Spar. Not deeply corroded. Several large blisters; otherwise in good condition.	167 AA–I. X. L. No. 2. Considerable blistering and corrosion. Coating easily scraped off.	Navy Yard, New York.
160 A–W. Z. in 30 K. Upper side perfect; lower side slightly blistered. Coating hard.	164 AA–20 K. Badly corroded; three-fourths of the varnish destroyed.	168 AA–Spar. Like 167, but not badly corroded.	
161 A–W. Z. in 40 K. No blisters; otherwise like 160 A.	165 AA–30 K. Like 164 AA.		
162 A–W. Z. in 30 Z. Like 161 A.	166 AA–40 K. Badly blistered, but not badly corroded. Coating on one side firm; on the other soft.	169 AA–D. M. C. Many blisters; very little corrosion; coating generally firm.	

TABLE No. 1—(*Continued*).

Lake Cochituate, Boston.	442–W. Z. in 20 Z. baked. Nearly perfect.	445–W. L. in 20 K. Very little corrosion. Some superficial blisters.	449–Um. Blue in 20 K. Considerable rust; not deep; paint practically destroyed.
	443–W. Z. in 30 Z. baked. Excellent; no rust; blisters superficial and few.	446–W. L. in 30 K. Good condition; no rust. Some superficial blisters.	450 Um. Blue in 30 K. A little worse than 449.
	444–W. Z. in Spar, baked. Like 443, or better.	447–W. L. in 40 K. Like 446.	451–Um. Blue in 40 K. Worse than 449; deep rust.
		448–W. L. in Raw Oil. Much deep corrosion; about half the plate in good condition.	452–Um Blue in Raw Oil. Like 451; whole surface rusted.
Navy Yard, Norfolk, Va.	195 AA–W. Z. in 30 K. Good; blistered a little on the edges.	199 A A–S. P. C. Blistered a little from edges; otherwise all right.	
	196 AA–W. Z. in 40 K. Fine, but discolored; like 185.	200 AA–S. P. C. Like 199.	
	197 AA–W. Z. in 30 Z. Fine, but blistered a little along the edges.		351–Um. Blue in 40 K. Nearly all gone.
	198 AA–W. Z. in Spar. Like 197.		352–Um. Blue in Raw Oil. Coating all gone; very badly rusted.
Navy Yard, New York.	170 AA–W. Z. in 30 K. Very little corrosion. Blisters amount to 1%. Coating good.	174 AA–S. P. C. In perfectly good condition.	
	171 AA–W. Z. in 40 K. Good, but not equal to 170 AA.	175 AA–S. P. C. Like 174.	
	172 AA–W. Z. in 30 Z. No corrosion; no blisters; excellent condition.		251–Um. Blue in 40 K. Very many small blisters; very little corrosion.
	173 AA–W. Z. in Spar. About like 172.		252–Um. Blue in Raw Oil. Uniformly corroded; coating all gone.

TABLE No. 1—(*Continued*).

453-Graphite in 20 K.	457-Keystone in 20 K.	461-Iron Oxide in 20 K.	Lake Cochituate, Boston.
Very good; some small blisters.	Good condition; no rust; some small blisters.	Very little rust; small blisters in outer coat.	
454-Graphite in 30 K.	458-Keystone in 30 K.	462-Iron Oxide in 30 K.	
Like 453.	Like 457.	Better than 461; no rust.	
455-Graphite in 40 K.	459-Keystone in 40 K.	463-Iron Oxide in 40 K.	
Like 453.	A little rust; many small superficial blisters.	Like 462.	
456-Graphite in Raw Oil.	460-Keystone in Raw Oil.	464-Iron Oxide in Raw Oil	
Deeply and generally rusted; about one tenth of the paint still good.	Badly and deeply rusted; patches of paint still good.	Corrosion deep and general; paint all gone.	
353-Graphite in 20 K.	357-Keystone in 20 K.	361-Iron Oxide in 20 K.	Navy Yard, Norfolk, Va.
Three-quarters gone; much rust.	Coating blistered and one-quarter gone.	Pretty good condition; a few blisters.	
354-Graphite in 30 K.	358-Keystone in 30 K.	362-Iron Oxide in 30 K.	
Half gone; much rust.	Blistered, but not destroyed	Not quite as good as 361.	
355-Graphite in 40 K.	359-Keystone in 40 K.	363-Iron Oxide in 40 K.	
One-quarter gone.	Blistered, but not in bad condition.	Like 361.	
356-Graphite in Raw Oil.	360-Keystone in Raw Oil.	364-Iron Oxide in Raw Oil	
Nearly all gone; badly rusted.	All gone; badly rusted.	Like 360.	
253-Graphite in 20 K.	257-Keystone in 20 K.	261-Iron Oxide in 20 K.	Navy Yard, New York.
A few blisters; very little corrosion.	No corrosion; numerous very small blisters.	Blistered, but not very badly. Not much corrosion.	
254-Graphite in 30 K.	258-Keystone in 30 K.	262-Iron Oxide in 30 K.	
Like 253.	Like 257.	Like 261. Not deeply rusted.	
255-Graphite in 40 K.	259-Keystone in 40 K.	263-Iron Oxide in 40 K.	
No corrosion. Paint in good condition. Numerous very small blisters.	Like 257.	Like 262.	
256-Graphite in Raw Oil.	260-Keystone in Raw Oil.	264-Iron Oxide in Raw Oil.	
Uniformly corroded; coating all gone.	Coating destroyed and plate badly corroded.	Like 260.	

TABLE NO. 1—(*Continued*).

Lake Cochituate, Boston.	465–Red Lead in Raw Oil. Paint still tough; looks well. Blisters from the bottom with slight corrosion beneath.	469–Eureka Paint. General corrosion; paint entirely destroyed.	
		470–Detroit Graphite. Like 469; paint nearly all destroyed.	475–International Holtzapfel. Like 469.
	467–Prince's Metallic in Raw Oil. About one-quarter deeply rusted; paint practically all gone.	472–Iron Oxide in Shellac mixture. Good condition; about 2% rusted.	477–Red Lead and W. Z. in Raw Oil. Many deep rust spots; about 5%; remainder good.
	468–Purple Oxide in Raw Oil. Like 467.		478–Red Lead and W. Z. in Raw Oil. Like 477. Not nearly as good as 465.
Navy Yard, Norfolk, Va.	365–Red Lead in Raw Oil. Coating destroyed; plate badly rusted.	369–Eureka Paint. Like 365.	374–McInnes' Paint. Like 372.
		370–Detroit Graphite. Like 365.	375–International Holtzapfel. Like 365.
	367–Prince's Metallic in Raw Oil. Like 365.	372–Iron Oxide in Shellac mixture. Paint destroyed; general, but not deep corrosion.	377–Red Lead and W. Z. in Raw Oil. Like 365.
	368–Purple Oxide in Raw Oil. Like 365.	373–Rahtjen's Paint. Like 365.	378–Red Lead and W. Z. in Raw Oil. Like 365.
Navy Yard, New York.	265–Red Lead in Raw Oil. Coating badly destroyed. Considerable corrosion.	269–Eureka Paint. Like 260.	274–McInnes' Paint. In good condition; no barnacles.
		270–Detroit Graphite. Like 260.	275–International Holtzapfel. Paint badly gone; much corrosion; many small barnacles.
	267–Prince's Metallic in Raw Oil. Like 260.	272–Iron Oxide in Shellac mixture. A few blisters; otherwise in good condition.	277–Red Lead and W. Z. in Raw Oil. Coating thin; gone in many places; considerable corrosion.
	268–Purple Oxide in Raw Oil. Like 260.	273–Rahtjen's Paint. Paint badly gone; considerable rusting. Many small barnacles.	278–Red Lead and W. Z. in Raw Oil. Like 277.

TABLE NO. 1—(*Concluded*).

481-20 K., baked. Practically perfect; coating still glossy.	484-30 M., baked. Several deep spots of rust, coating badly blistered.	487-I. X. L. No. 2, baked. Like 481.	Lake Cochituate, Boston.
482-30 K., baked. Like 481.	485-30 Z., baked. Like 481.	488-Raw Oil, baked. Badly and deeply rusted. Two-fifths of the surface good.	
483-40 K., baked. Like 481.	486-Spar, baked, Like 481.	489-D. M. C., baked. Fine; a few small blisters in the outer coat.	
381-20 K., baked. Half of the coating destroyed; the rest good. Not much rust.	384-30 M., baked. Like 382.	387-I. X. L. No. 2, baked. Like 386.	Navy Yard, Norfolk, Va.
382-30 K., baked. Four-fifths destroyed; very little rust.	385-30 Z., baked. Like 382.	388-Raw Oil, baked. All gone; rusted.	
383-40 K., baked. Like 382.	386-Spar, baked. Nearly all gone; little rust.	389-D. M. C., baked. Three-quarters gone; remainder good; very little rust.	
281-20 K., baked. Plate thinly rusted along the edges.	284-30 M., baked. Many small and some medium-sized blisters. Not badly rusted.	287-I. X. L. No. 2, baked. Not much corrosion; very small blisters.	Navy Yard, New York.
282-30 K., baked. Small blisters, with thin rust beneath, over most of the plate.	285-30 Z., baked. Good condition. Very little rusting. Very small blisters.	288-Raw Oil, baked. Badly corroded. Coating destroyed.	
283-40 K., baked. Very small blisters; not much rust.	286-Spar, baked. Coating badly destroyed; much corrosion.	289-D. M. C., baked. Very many small blisters. Not very much corrosion.	

DISCUSSION.

Mr. Buck. L. L. BUCK, M. Am. Soc. C. E.—Have any of the specimens been tested by being placed in salt water so as to be exposed part of the time, and part of the time not exposed?

Some tests of paints on wires were made for the speaker by E. G. Freeman, M. Am. Soc. C. E. The wires were about No. 8 gauge and cut in lengths of about 6 ins. They were coated with different kinds of paint and were then fastened together snugly in bundles of seven. The interstices were filled and the bundles coated with paint outside. These bundles were fastened to a pile, under the resident engineer's office, midway between high and low water. They were protected from abrasion by floating matter, and were left in position for six months, but not in freezing weather.

The reason for making the test in this way was that the conditions obtained would approach, more nearly than any others, the conditions which would be met by the cables of the New East River Bridge. The wires were found to be corroded even inside, where the interstices had been filled with the paint. There were only one or two substances which stood the test. One, called "cable shield," the composition of which the speaker knows, but is not allowed to divulge, withstood abrasion and corrosion perfectly. This material was selected as the covering of the cables, although its application was somewhat difficult.

Has the author used Lucol oil in any of his experiments?

The speaker has had some trouble in having paint applied properly. Having secured an honest manufacturer and obtained from him the required paint, he has found that a further requisite was the right kind of a man to apply it. The speaker has had structures painted and has found that after six months the pigment could be rubbed off with the hand quite readily. In the next specification prepared by the speaker he has concluded to insert a clause requiring the paint manufacturer to have an inspector present all the time to see that the paint is properly put on. In this way it may be possible to fix the responsibility if the work is not satisfactory.

Mr. Hill. GEORGE HILL, M. Am. Soc. C. E.—The author has said that he considers these tests a fair criterion by which to judge of the sufficiency of the coating, as applied to structural material exposed to the air or the elements. What time relation is considered by him as existing between the exposure to fresh or salt water and the exposure which would exist, for instance, in the steel skeleton of a building surrounded with masonry? That is to say, if after an exposure of two years in water and one or two years to air most of the coating has dis-

appeared, how many years will it take to produce a like result if the material is partially protected from corrosion? Mr. Hill.

Further, would all the protective coverings show the same relative deterioration under the conditions existing in buildings as they show when subjected to the direct action of fresh and salt water?

During the past six years the speaker has had Lucol oil under observation, and has used it quite extensively (for the last three years exclusively for exterior work and metal covering), and has found that it gave better results than any of the other oils. The speaker would like to ascertain the author's general views in regard to oil as compared with varnish. If the author's statements in regard to recent structural work in New York City are taken without qualification, it may be said that the life of the largest, or best, office buildings will be about twenty years, if the conditions under which the exterior supporting columns are installed are such as to subject these columns to a certain exposure; that is to say, if they are placed with their outer edges from 8 to 24 ins. from the exterior walls. The general practice is not to make any special provision for protecting the columns.

The speaker recalls one case in which it was specified that the columns should be left with a space around them. This space between the column and the enclosing brickwork was to be filled in subsequently with Portland cement, which would be, without doubt, a sufficient protection if the column were not otherwise protected; but with a film of paint between the metal and the cement the speaker doubts the sufficiency.

In New York City the practice, which is nearly universal, is to use one of the paints or, occasionally, Smith's durable metal coating, but if these have a life of approximately only twenty years, the New York practice should change quickly.

The paper does not indicate what should be done, yet some lesson should be derived from these tests to show what the practice ought to be, for engineers do not want to build for only ten, fifteen or even twenty-five years.

F. W. Skinner, M. Am. Soc. C. E.—In New York City the general practice is to protect from moisture quite effectively certain parts of the steel work of office buildings. Most of the grillage foundations, for instance, are bedded in concrete, and frequently the metal surfaces are also plastered. The wall columns, in many cases, are only protected by paint, but in other cases there is a special protection. Mr. Skinner.

In the St. Paul Building, for instance, there is a tile casing around the columns and also a coating of asphalt, or some kind of fibrous material wrapped around them.

Thomas D. Pitts, Jun. Am. Soc. C. E.—The author states that in these experiments the plates exposed in fresh water were placed vertically, while those exposed in salt water were placed horizontally. Mr. Pitts.

Mr. Pitts. Was there any special reason for so placing the latter? There would necessarily be a deposit of silt on the upper surfaces of these plates which would measurably protect the coating. It would also prevent the growth of oysters, barnacles, seaweed, etc., which will not grow on surfaces where they cannot get a firm hold. Moreover, the silt deposit would not necessarily be uniform on all the plates, so that they would not all have the same degree of protection.

This being true, would not more uniform conditions be secured by placing all the plates vertically?

Mr. Tatnall. GEORGE TATNALL, M. Am. Soc. C. E. (by letter).—The subject of protective coatings for iron is one of great depth, the surface of which has been little more than scratched. The universal use of iron, for such diverse purposes divides the subject into a number of problems according to the exposure to which the iron is subjected.

The protection of ironwork exposed to ordinary out-of-door conditions of sunshine, rain, dew and variations of temperature is the most usual problem. The protection of ironwork in the indoor exposure incident to the roofs of trainsheds, foundries, shops and other manufacturing establishments, presents another and very different problem, wherein the absence of the direct heat and light of the sun, and the drenching of rain, is offset by the presence of deleterious and corrosive gases. The protection of the skeletons of steel-framed buildings forms a third distinct problem.

Two other very similar problems, and very dissimilar to the others, are the protection of ironwork submerged in water, and ironwork alternately submerged and exposed, as by the fluctuation of tides.

These problems present such dissimilar features that the results of tests under one, can by no means be taken as more than an indication of possible results under one of the others.

This paper presents some valuable and instructive data relative to the protection of iron completely submerged.

Charles B. Dudley, M. Am. Soc. C. E., Professor Spennrath and others have shown the effect of submersion in water on a linseed oil film, manifested in the softening, wrinkling and loss of adhesiveness of the film. This receives abundant confirmation in the blistering and separation of the coats, so frequently noted.

By far the greatest enemy to paint coatings on iron is the well-known and uncontrollable propensity of the metal to rust when in the presence of oxygen and moisture, which is so fierce as to produce, by some sort of endosmotic and exosmotic action, the rusting of the iron and deposition of the oxide on the outside, through the pores of protective coatings of other metals, such as tin, zinc or lead. Bright iron will not rust in an atmosphere of pure oxygen, nor will it rust when immersed in distilled water, and it is questionable whether the scanty amount of air ordinarily held in suspension by river or sea water would

be sufficient to make the test as severe as it would be under conditions part wet and part dry, or even under those of ordinary outdoor exposure. In support of this, it can be stated that similar plates, to the knowledge of the writer, covered with two coats of some of the same paints mentioned in these tests, passed to complete destruction in from 6 to 12 months in ordinary outdoor exposure. Mr. Tatnall.

Almost invariably, the protective value, to iron, of a good paint, is destroyed in outdoor exposure by the penetration of moisture and air through the pores of the coatings; causing the formation of rust spots, microscopical at first, but increasing in size and number, and spreading and joining together under the film, until it is thrown off in flakes or shreds, long before its good qualities as a paint have disappeared. On this account, the very able prize essay of Professor Spennrath, complete and exhaustive as it is, in relation to the paint film, is useless as regards the protective value of that film to the iron beneath it.

The large number of samples, in these tests of submerged exposure, in which it was noted that the coatings had been destroyed and very little or no rusting manifested, would seem to indicate that the processes of destruction were different in this than in other exposures. The film seems to have been destroyed before the rusting commenced, but how? The almost invariable presence of blisters, or separation of the two films from each other and from the metal, shows conclusively the same softening, loosening, and wrinkling action on these films, as occurred in the case of linseed oil films on glass submerged in water, in the experiments of Dr. Dudley. But it has certainly not extended so far as to effect complete separation of the films from each other or from the metal, or some half-peeled remnants would have been found.

The absence of any explanation for this rustless destruction of the film in submerged tests, as well as certain unexplained phenomena occurring in ordinary outdoor exposure, suggests the query, does linseed oil, by complete oxidation, by long submersion, by the presence of the naturally occurring chemical reagents, or by the combination of any or all of these causes, become to any extent soluble in water?

Although the varnish gums are exceedingly unreliable and uncertain, whether used on wood or iron, in ordinary exposures, it seems to be shown clearly by these tests that the addition of the best of them to linseed oil is of great benefit to the latter in resisting the soaking destruction of submersion, by whatever influences this may be caused.

Oscar Lowinson, Assoc. M. Am. Soc. C. E.—The speaker has made a number of tests of a new paint, and with a view to determining its resistance to moisture, and the effects of frost, a coat was applied to the outside surface of a porous jar which was afterward filled with a concentrated solution of sulphate of soda and the solution allowed to evaporate. After a period of two weeks the interior of the jar was full of crystals, and with the exception of half an inch from the top of Mr. Lowinson.

Mr. Lowinson

the outside, which had been carried over by capillarity, the surface was still intact and showed no signs of having in any way been penetrated. This experiment was repeated with the same jar half a dozen times with the same results. A corresponding jar to which no paint was applied emptied in about 30 hours, both the inside and outside being covered with sharp crystals.

In order to determine whether this material could be used as a substitute for furring, a brick coated with it on its top surface was plastered directly on the paint and immersed to one-half its depth in a vessel containing water, until all the water, including that which the brick absorbed, had been evaporated. This process took a week, and showed absolutely no effect on the plaster. The plaster adhered so strongly to the paint that it could not be dislodged without destruction.

A similar brick was plastered without paint, and in a week the plaster became loose and detached from the brick.

"Furring," which is used on the inside of exposed walls of buildings to prevent moisture from attacking the plaster and thereby disintegrating it or staining the finished wall surface, can be dispensed with by the use of this paint on the wall surface and the direct application of the plaster upon it. The speaker also uses it on the back of limestone set in masonry to prevent the discoloration of the wall, which occurs when any other cement than the expensive LeFarge is used.

The paint also appears to resist the attacks of moderate acids and alkalies, and herein differs apparently from the paints mentioned by the author.

Two coats were applied on a rusty, cast-iron, fire stand-pipe in front of a building in New York City where it has stood for a year without flaking or showing rust spots, and when last examined by the speaker it was still hard and adherent, and to all appearances had stopped oxidation.

The speaker has not been converted to the theory of the author that laboratory tests are not of much value. He firmly believes in exposure tests, but, in order to satisfy himself, deems it his duty, when examining a material like paint, to make such tests as he thinks would give an imitation of the action of the elements.

Mr. Sabin.

A. H. SABIN, Assoc. M. Am. Soc. C. E. (by letter).—None of the tests in this series was made with the samples alternately wet and dry. Some experiments, however, were made by the writer by coating bundles of wires, which were then attached to piles in the sea water in such a way that one end of each bundle was under water constantly, while the upper parts were exposed to the air at low tide. The conclusion reached from this and other experiments was, that the protective effect of the coatings was not lessened by the intermittent

exposure, but that damage was done by floating objects in the water which persistently batter off the coating at the water-line. If secured against such effects protection is possible. This is probably the reason why iron piles rust off near the water-line; they are constantly battered by logs of wood and other floating matter which the waves throw against them incessantly. Mr. Sabin.

One of the objects sought by placing the plates horizontally in salt water was to get a growth of marine organisms on one side of the plate, while the other (the upper) side was kept free and reasonably clean. There was no considerable deposit of silt on these plates. In this way the exact effect of the organisms could be determined. The plates put in fresh water were laid in a vertical position because it was unavoidable under the circumstances, but as fresh water organisms are comparatively rare and did not attack the plates, it was no serious objection. The most serious trouble in these experiments was the injury to the coating on the edges of plates caused by floating objects in the water. This was sufficient, in some cases, to obscure the real value of the coatings, which were removed mechanically from the edges toward the middle of the plates. The writer is of the opinion that, hereafter, plates to be tested, which should not be in any case less than $\frac{1}{8}$ in. in thickness, and should not be smaller than those used in this test—the larger the better—should each be put in a frame such as used for the slates of school children, in order to protect the edges as well as possible.

It is probably impossible to give any definite answer to Mr. Hill's question. The exposure in steel frame buildings is extremely variable. Good compact hydraulic cement mortar, unquestionably, affords a great deal of protection, because it is alkaline, and alkalies do not rust iron; and also because it prevents much circulation of air, though it does not do so absolutely. Ordinary lime mortar is so very porous that its value is much less; and brick and tile vary so much in porosity that no generalizations about them can be made. The care with which these are applied is also an important factor. It seems to the writer that the most important object to be sought in these cases is to keep the metal dry. It is not possible to keep a bridge or similar structure dry, but it is possible with the steel frame of a building; if it is dry it ought not to rust very fast. Foundations which are so placed as to be always damp should have their steel work bedded in cement, which should be of as perfect quality and as great thickness as possible.

The writer does not share in the prejudice which some have against painting metal which is to be covered with cement; it is not likely that the latter has any mysterious action on the iron, but acts as any other protective coating does; and it is well known that some paints which do not bear ordinary exposure last well under cement.

Mr. Sabin. The writer has not used Lucol oil, and hence has nothing more substantial than a prejudice against it. He has preferred to keep to the use of materials of known composition, and which he thinks have yielded the best results up to the present time. It is well known, as has been pointed out by Mr. Tatnall, that oil-paint films are somewhat porous, and are often thrown off by corrosion underneath; and it has been the writer's aim to produce films which would not be open to that objection, but would protect the metal until the film itself is completely destroyed. The object of this paper is to show how this may be accomplished by the use of well-known materials, combined by well-known processes.

The writer cannot at all agree with the statement that the use of varnishes is a matter of great uncertainty, but, in fact, believes the opposite, and desires by these tests to show what products are most suitable for use on engineering structures. Furniture made 150 years ago by the celebrated Martin, of Paris, is still distinguished by its beautiful varnish; while violins, ranging from 200 to 300 years old, are still covered with the varnish originally applied to them. Such material is worthy of systematic study.

Vol. XLIII. JUNE, 1900.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

No. 875.

THE FOUNDATIONS OF THE NEW CROTON DAM.

BY CHARLES S. GOWEN, M. Am. Soc. C. E.

PRESENTED FEBRUARY 21ST, 1900.

WITH DISCUSSION.

In 1883 the Legislature of the State of New York passed an Act (Chapter 490, Laws of 1883) creating the Aqueduct Commissioners of the City of New York.

The purpose of this Act was the immediate increase of the water supply of the city which, under the conditions then prevailing, had for some time been inadequate and inefficient. To this end it was planned to begin the construction of a new aqueduct and a large dam on the Croton River, the latter near to and above the site of Quaker Bridge, at a point about 4 miles below the old Croton Dam which had been in use since 1839. This new dam, it was reckoned, would increase the available storage by about 32 000 000 000 galls., and, if construction were begun immediately, could be put to practical use, in connection with the New Aqueduct, not long after the completion of the latter, the work of which was planned to continue at the same time.

The Aqueduct Commissioners began the construction of the New Aqueduct in the fall of 1884, but found a strong opposition, on the part of a few influential citizens, to the project of the dam. This opposi-

tion resulted in an indefinite delay in action on the part of the Commissioners, so far as the large dam was concerned, but they ordered the construction of a smaller dam and reservoir near the head waters of the East Branch of the Croton, at the Village of Sodom, early in 1888. This action reversed the original plans for the enlargement of the water supply, which were, to build the large dam and basin at first and with as much speed as practicable, and later to complete the conservation of all the storage capacity of the Croton Valley by building the smaller dams and reservoirs, of which the dam at Sodom was one.

In July, 1888, a new Board of Aqueduct Commissioners came into power. They found a steadily increasing demand for more water, and came to the conclusion that it was best to continue the policy of building the smaller dams and reservoirs already inaugurated by their predecessors, as, owing to the time which had lapsed (about 4 years), without action relative to the proposed large dam, it was impossible, even by taking immediate action toward its construction, to complete it in time to afford the desired relief in the water supply. They, therefore, ordered the construction of the Carmel Dams (Reservoir D) and the Titicus Dam (Reservoir M), as well as the completion of the Sodom Dam System and Reservoirs, which included two dams, two reservoirs and a connecting tunnel. The construction of these works was started as soon as practicable, and further investigations were authorized in relation to the proposed large dam, in order to determine whether the best available site had been found.

To this end an extensive series of diamond-drill borings was made along the valley of the Croton River from the site of Old Croton Dam to a point nearly at the mouth of the river, about 1 mile below the old Quaker Bridge site. The result was the decision of the Commissioners, in January, 1891, to build the large dam at the Cornell site, a point about 1¼ miles above Quaker Bridge, and so situated as to store nearly as much water as would have been stored by the Quaker Bridge Dam. The amount of storage by the dam if built at the Quaker Bridge site is estimated at 32 000 000 000 galls.; at the Cornell site, 30 000 000 000 galls.

In connection with these new dams and storage reservoirs are various older dams and natural lakes, throughout the water-shed of the Croton, which have been in use for the city's water supply for many

years, in connection with the Old Aqueduct; and the total storage capacity, upon the completion of the New Croton Dam, will be as follows:

Total storage in connection with the old works, including Central Park, Boyds Corners and Middle Branch Reservoirs..	9 541 000 000	galls.
Amawalk Dam.....	7 000 000 000*	"
Reservoir I, Sodom and Bog Brook Reservoirs.....	9 028 000 000	"
Reservoir D, Carmel.....	9 000 000 000*	"
Reservoir M, Titicus.....	7 167 000 000	"
New Croton Dam Reservoir.....	30 000 000 000*	"
Jerome Park Reservoir.....	1 500 000 000	"
	73 236 000 000	"

As the large reservoirs within the city territory cannot be emptied below certain limits without impairing the supply, the available storage capacity may be stated as about 70 000 000 000 galls.†

The construction of the New Croton Dam was begun in October, 1892, the contract for its construction having been let the preceding August. At the present time it is about two-thirds completed, and, as a general description of the structure, embodying its main features, is essential to the purposes of this paper, the following extracts from the "Report of the Chief Engineer, A. Fteley," Past-President, Am. Soc. C. E., "to the Aqueduct Commissioners, 1887 to 1895," are reprinted here, as they seem to embody the main points and important features in comparatively few words.

"The New Croton Dam at Cornell Site which is to form the largest reservoir of the system, on the lower part of the Croton River, was begun in October, 1892. It is located about 3¼ miles above the junction of the Croton with the Hudson, and about 1 mile above Old Quaker Bridge. The course of the Croton at this point is approximately west.

"At the dam location, rock (gneiss) crops out at the surface on the north side of the river, rising with a steep slope, which terminates at the top of a hill about 300 ft. high. The bed-rock on the north side dips quickly just before reaching the bank, and soundings show it at about 75 ft. below the river-bed. At this point, on a line about

*Approximate.

†Report of the Chief Engineer to the Aqueduct Commissioners, 1887-1895, p. 82.

parallel to and under the river, the rock changes abruptly from gneiss to limestone, with no marked change of surface level. The limestone extends across the valley at about the depth noted above, with some deeper pockets, and then rises gradually on the south side with the earth slope and below it, at varying depths, to a depth of 20 ft. at the extreme south end of the dam location.

"Under the river-bed the material above the bed-rock is largely sand, gravel and boulders. Approaching the south side of the valley, very compact hardpan and gravel next to the rock is indicated. The hardpan is surmounted next to the surface by a considerable layer of sand at the steep part of the slope. Along this slope, at about elevation 153 runs the Old Croton Aqueduct. The total distance across the valley at flow-line (elevation 200) is about 1 300 ft.

"The general features of the dam may be noted as follows:

"*An overflow, or spillway*, on the rocky side-hill forming the north slope of the valley.

"*A masonry dam* built on bed-rock and extending from its junction with the overflow at about the foot of the north slope of the valley, across and well into the south slope, where it ends in a wing-wall and core-wall for the embankment.

"*An embankment* with a core-wall extending to bed-rock from the end of the masonry dam up and along the south slope until elevation 220, the proposed top of this part of the dam, is reached.

"*The overflow* varies in height from 150 ft. at its junction with the main dam to about 10 ft., where it joins the side-hill at the upper end. This overflow runs along the side-hill and nearly parallel to the slope contours, curving up-stream at its junction with the masonry dam. The down-stream face of the overflow is to be formed in steps. From the spillway the water is to fall into a channel cut into the rock of the side-hill, through which the water will pass on its way to the river-bed below the dam. This overflow channel is to be about 50 ft. wide at the upper end and 125 ft. wide next to the main dam. The length of the overflow will be nearly 1 000 ft., elevation of top, 196.

"*The masonry dam* will extend from bed-rock to elevation 210, and provision is made for a roadway on top, 18 ft. in width. At the north end, near its junction with the overflow, is to be a gate-house of three chambers. Grooves in the masonry of the up-stream face will be provided for stop-planks, and in each chamber will be gates worked from the top of the dam, connecting with a 48-in. pipe. The pipes will extend through the dam, ending in a vault, containing stop-cocks to further control the flow of water. It is expected to place the lower openings in the gate-chambers at about elevation 75, nearly 30 ft. above the original river-bed, and to fill in this interval with earth, forming an embankment with a flat slope above the restored original surface, on the up-stream side.

"The masonry dam will be about 600 ft.* in length from its junction with the overflow to the back of the wing-wall at the south end, and its extreme height will be 260 ft. or more, as the soundings show some large and deep depressions in the rock surface below. Maximum thickness at bottom next to rock, about 190 ft.

"The embankment extending south from the wing-wall end of the masonry dam will have a core-wall extending throughout its length, founded on bed-rock, thus forming with the overflow and main dam a continuous masonry connection with bed-rock throughout the whole length. From elevation 64 down to bed-rock this wall is to be not less than 18 ft. in thickness; from elevations 64 to 200 the wall gradually diminishes to 6 ft. in width at the top. The elevation of the top of the embankment is 220; width at top, 30 ft. Up-stream slope, 2 to 1, paved; down-stream slope, 2 to 1, broken with three berms, each 5 ft. wide at different elevations. These berms will be ditched and paved to carry rain-water from the slopes, which are to be soiled and sodded.

"The Old Aqueduct is discontinued between the slope lines of the embankments, and is being replaced by a new section built on a curved line into the side-hill, nearer the extreme south end of the dam. At the junction of this new line of Aqueduct with the core-wall masonry, a second gate-house will be built for the purpose of connecting the water impounded in the New Reservoir with the Old Aqueduct.

* * * * * * *

"The gate-house foundation rests on bed-rock, and the curved line of the new section of the Aqueduct was designed to avoid the deep excavation for this foundation, which would have been necessary had the original location on the Old Aqueduct line been adhered to. The gate-house is drained by a system of 12-in. pipes, which are connected with the bottom of each chamber and unite into one pipe laid under the invert of that part of the new section of the Aqueduct lying on the down-stream side of the core-wall. Near the junction of the New Aqueduct Section with the Old Aqueduct, this drain pipe, after a short turn, emerges in the adjacent hillside.

"The center of the overflow and masonry dam, the core-wall, the gate-house foundations, the side walls of the Aqueduct, the backing of the gate-house chambers and inlet conduits will be built of rubble masonry. The overflow will be faced above the surface of the ground with coursed facing-stones cut to specified rises. On the down-stream side the steps are to be laid with block-stone masonry generally heavier in rise and width than the facing-stone, and of depth sufficient for a bond under the next step above.

* * * * * * *

* This length has since been increased to 710 ft.

"The main dam and the outer faces of its gate-house will be faced, wherever exposed, with facing-stone of the same class as that in the overflow.

* * * * * * *

"For the protection of the deep earth excavation, which is to take place in the bottom of the valley for the foundation of the dam, the river is diverted from its bed for a distance of over 1 100 ft. For that purpose an extensive rock cut has been made into the north side-hill and the river has been turned into this new channel" (125 ft. in width) "which is formed on the river side by a substantial river-wall founded in rock.

"This wall, parallel with the old river-bed and 600 ft. long, is connected at both ends with temporary wing-dams extending across the valley, above and below the site of the dam, thus making a complete inclosure, inside of which the excavation can take place without interference from the river. The wing-dams are built of earth with a timber core formed of two thicknesses of plank tongued and grooved, each 3 ins. in thickness. The timber core extends to a depth of 20 to 25 ft. below the natural ground. The toe of the dams on the river side is formed by heavy crib-work, intended to break the force of the current in time of freshet. The toe of the lower wing-dam is further protected by sheet piling and by a heavy weight of rock to counteract the erosive action of the large flow which may be discharged from the new channel into the river in case of a heavy freshet.

* * * * * * * *

"The total length of the protective work just described, including the river-wall and the wing-dams, is 1 600 ft. The capacity of the new channel has been designed to safely accommodate a flow equal to the largest freshet recorded in Croton River since the construction of the old works, when the discharge was approximately 15 000 cu. ft. per second."

In connection with this description, attention is called to Plate XXXV, which is an outline plan of the structure and shows, in addition to the various features noted above, the outline of the excavation necessary for the main dam foundation masonry, and the embankment to be built against the core-wall with which it forms the south end of the structure.

Figs. 1 and 2 show various sections of core-wall and embankment, of the main masonry dam at various points and the maximum section of the overflow wall where it crosses the temporary river channel.

The dam was designed and its construction is being superintended by Mr. Fteley, the Chief Engineer. He was assisted, for the mathematical computations necessary for determining the main section, by

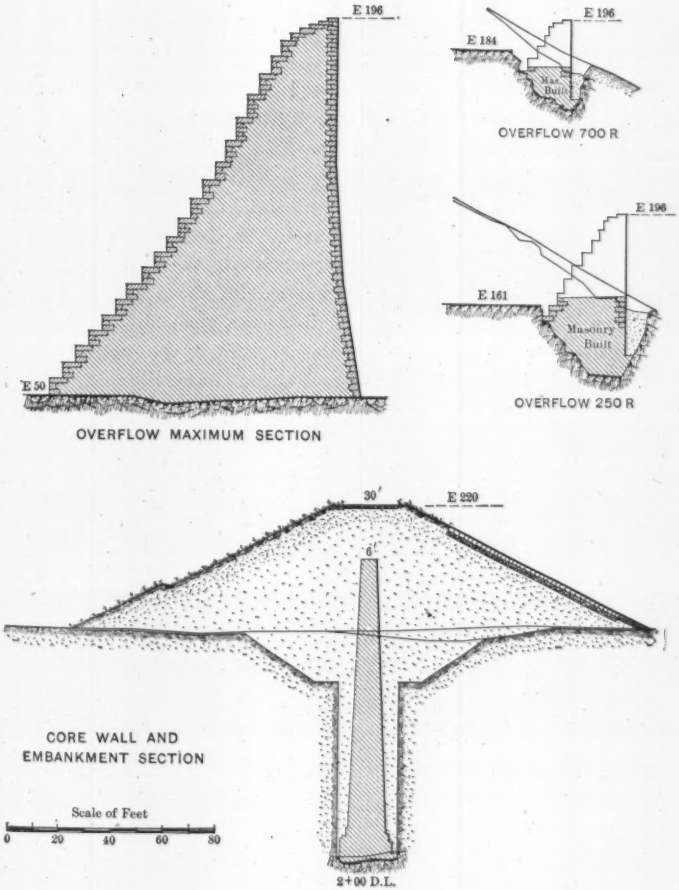

E 196
E 50
OVERFLOW MAXIMUM SECTION
E 196
E 184
Masonry Built
OVERFLOW 700 R
E 196
E 161
Masonry Built
OVERFLOW 250 R
30'
E 220
6'
CORE WALL AND EMBANKMENT SECTION
Scale of Feet
0 20 40 60 80
2+00 D.L.

FIG. 1.

E. Wegmann, M. Am. Soc. C. E., who has since developed and formulated the methods followed, in his book on high masonry dams.*

It may be said that the section adopted affords a factor of safety of 2 against any tendency toward the overturning of the structure.

The work of construction has been conducted, from its inception, under the immediate direction of the writer.

Since the foregoing description was written, the protective work has been completed, substantially as outlined. The earth and rock excavations for the foundations have been finished; the foundation masonry practically all laid, excepting a short stretch of the overflow which is to cross the river-channel cut and join the long stretch of overfall foundation masonry already laid. The length of this remaining stretch is about 250 ft. In the progress of the above work the section of the main dam masonry was carried about 110 ft. further into the side-hill at the south end than was planned at first, thus shortening the core-wall and embankment section by the same distance, and, owing to the rise in the bed-rock surface under the south slope, decreasing the maximum depth or height of the core-wall and embankment considerably from that of the original design.

Owing to the character of the limestone, which rendered deep excavation necessary at certain points, the extreme height of the masonry dam will range from Elevation—80, the lowest point reached in the foundation excavation, to Elevation 210, a total of 290 ft. For the same reason the extreme thickness of the main dam masonry at the toe is about 200 ft.

Borings.

The final location of the New Croton Dam resulted from the indications furnished by an extensive series of diamond-drill borings, during which the Croton Valley was explored thoroughly along the line of the river from an old mill at the head of tide water, about three-quarters of a mile below Quaker Bridge, to Old Croton Dam, a distance of about 5 miles. The general system for determining upon the position of the borings proposed, was as follows: Whenever the appearance of the surface seemed to be favorable a number of drill holes were made on a line parallel with the river, and, if one of them gave indication of the proximity of bed-rock to the surface, a transverse line of holes was drilled across the valley at that point.

* "High Masonry Dams," by E. Wegmann, Jr., M. Am. Soc. C. E., New York, John Wiley & Sons, 1891.

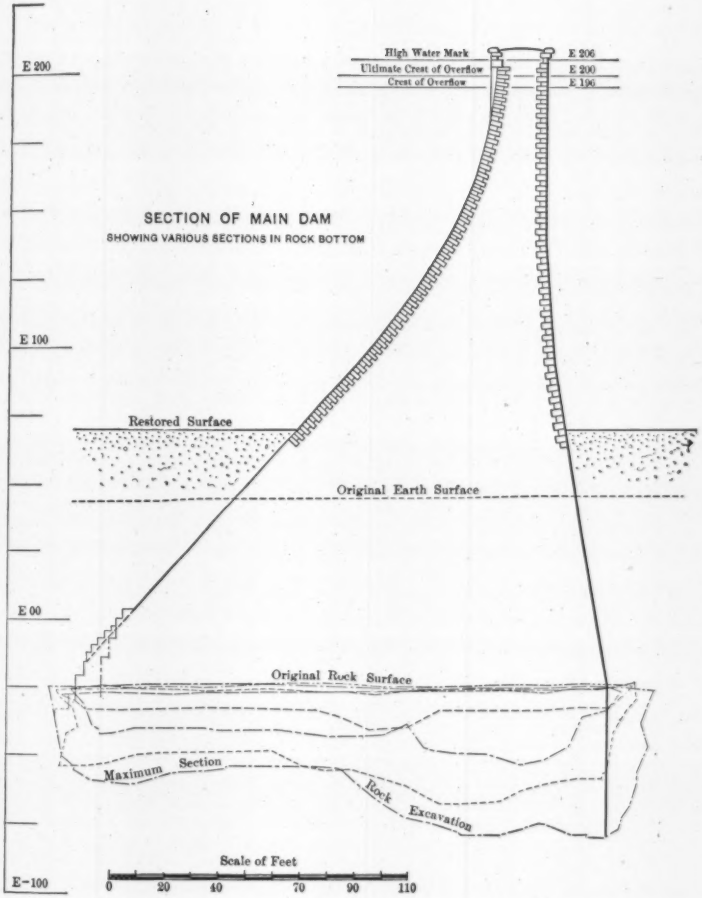

E 200
E 100
E 00
E−100
High Water Mark
Ultimate Crest of Overflow
Crest of Overflow
E 206
E 200
E 196
SECTION OF MAIN DAM
SHOWING VARIOUS SECTIONS IN ROCK BOTTOM
Restored Surface
Original Earth Surface
Original Rock Surface
Maximum Section
Rock Excavation
Scale of Feet
0 20 40 70 90 110

FIG. 2.

In this way a large number of transverse lines was drilled, and it was found almost invariably that wherever the bed-rock cropped to the surface on one side of the valley it dipped down sharply on the other side to a depth at which, in most cases, it would be impracticable to establish a foundation. As a rule, gneiss was found, but at various points on either side there are formations of limestone with clearly defined points of separation which, in some cases, were under the bed of the stream.

Several of the more favorable locations thus indicated were explored more particularly by a number of transverse lines of holes about 100 ft. apart, and when the present established location was finally determined upon, additional borings were made, to cover the site of the masonry structure, at intervals of 50 ft. Fig. 3 shows the location and result of these borings, as well as the outline of the proposed foundations. It will be seen that there were in places indications of a considerable depth of soft white rock (partly disintegrated limestone), before the hard rock was reached, extending in one case to an extreme of nearly 40 ft. The holes drilled in the rock were, as a rule, 2 ins. in diameter, and were carried to a depth presumably sufficient to establish the character of the rock below. The hard white rock sought for, and, as a rule found before the borings ceased, was mostly bluish limestone, while the soft white rock varied in its texture from white limestone, friable under some pressure, to very friable or wholly decomposed rock. The line of separation between the limestone and the gneiss was shown to be directly under and parallel to the river-bed. The borings indicated further, the presence of seams, more or less open, in the limestone, and the frequent reports of the sudden loss of the water (*i. e.*, the water supplied by the steam pump to wash out the holes as the borings progressed) showed that these seams were connected in places with rather free flowing outlets. As the general level of the bed-rock was at Elevation —25, or about 75 ft. below the river, and as the water table in the sand and gravel above this bed-rock was substantially the same as the river level, it is perhaps a question of some interest as to how and where this disappearing stream went, and, in case of its reappearance, what were the causes which may have led to it. Copies of the drill runner's log, which follow, show the records of Holes Nos. 95 and 99.

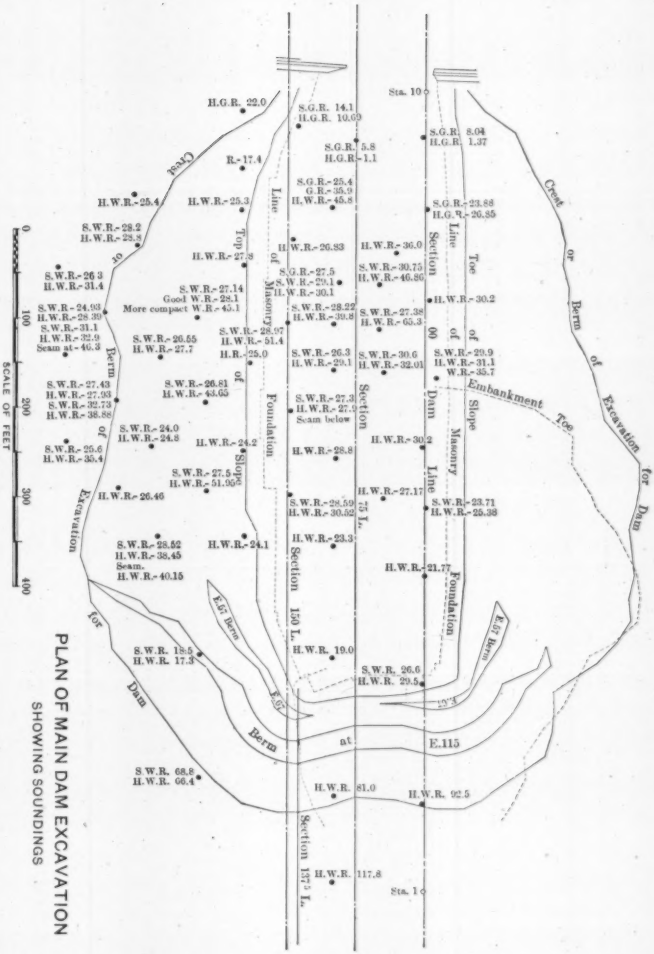


FIG. 8.

PLAN OF MAIN DAM EXCAVATION
SHOWING SOUNDINGS

These furnish two illustrations out of a number of cases in which the water disappeared and reappeared after an interval. Hole No. 99 is especially noticeable, as the final disappearance of the water did not occur until the drill had reached its lowest level, Elevation —76.80.

HOLE NO. 95.—Elevation of Jack Plank..... 71.9
" " Ground..... 69.6

Date. 1892.	Material.*	Depth, below Jack Plank.	Remarks.
April 20.	S. & B.	7.23	
" 21.	"	27.50	
" 23.	.....	.....	Broke casing; third joint up 28.35.
" 22.	.....	.....	Pulled out; got back to 28.35.
" 22.	Boulder.	28.35	X bit 10 ins. below shoe.
" 22.	C. S. & B.	30.60	
" 25.	"	36.00	Don't stand up; fills in.
" 26.	"	43.25	" " "
" 27.	"	51.00	" " "
" 28.	"	56.00	" " "
" 28.	S. G. S.	58.00	Very little flow as soon as X bit is below shoe.
" 29.	"	59.00	Very little flow as soon as X bit is below shoe.
" 29.	H. S. & S.	74.00	Stands up and fills in; can pound down; stands up; no flow.
" 30.	"	79.00	Stands up and fills in; can pound down; stands up; no flow.
May 2.	"	91.00	Stands up and fills in; can pound down; stands up; no flow.
" 3.	"	94.00	Fills in very bad; cannot get powder down.
" 4.	"	98.67	Stands up good.
" 5.	"	100.87	—28.97 top of soft white rock.
" 5.	S. W. R. & Sand.	104.85	—32.95 I think this is fine sand; the floor was clear.
" 5.	S. W. R.	105.90	No core.
" 6.	"	106.90	Lost flow.
" 6.	"	110.00	Flow came back; no core.
" 6.	"	111.00	The rock is a little harder; no core.
" 6.	"	114.85	Not hard enough to core.
" 6.	"	118.85	" " " yet; no core.
" 6.	"	122.00	Lost flow.
" 6	"	122.95	Not hard enough to core; no core.
" 6.	"	123.30	Commenced to core —51.40.
" 6.	H. W. R.	124.55	0.90.
" 6.	"	126.95	0.90.
" 6.	"	130.25	1.80 Elevation of water in casing x 45.9.

* S. & B. —Sand and boulders.
C. S. & B.—Coarse sand and boulders.
S. G. S. —Sand, gravel, stones.
H. S. & S.—Hard sand and stones.
S. W. R. —Soft white rock.
H. W. R. —Hard white rock.

"This hole is the same as Hole No. 88; stands up very good, but could not go far below the shoe, the flow would go away. The rock from elevation —28.97 —51.40 was very soft, but stood up very good, and did not cave in, if it had I could not have drilled so far down. W. J. S. (Signed) Tierney, Foreman."

HOLE No. 99.—Elevation of Jack Plank.....72.2
" " Ground.....70.0

Date. 1892.	Material.*	Depth, below Jack Plank.	Remarks.
May 19.	S. & S.	15.00	Moved, set up, down to 15.00.
" 20.	F. S. & S.	36.50	Loose fine sand and no flow.
" 20.	C. S. & S.	42.86	Stands up good, flow came back.
" 20.	"	51.00	" " "
" 23.	"	53.00	" " "
" 23.	S. G. & S.	64.50	Telescoped with 4-in. casing to 38.50.
" 24.	"	73.00	" 2½-in. "
" 25.	"	79.00	Fills in bad.
" 26.	"	88.00	"
" 27.	"	91.50	"
" 28.	"	95.00	Stands up good, very stony.
" 30.	"	97.50	" "
June 1.	"	99.58	Top of S. W. R. —27.38.
" 1.	S. W. R.	100.80	
" 1.	"	103.0	Put in diamond bit.
" 1.	"	104.9	No core.
" 1.	"	108.4	"
" 2.	"	114.1	"
" 2.	"	121.8	"
" 2.	"	125.9	"
" 2.	"	128.2	"
" 2.	"	130.5	Commenced to core —58.30.
" 2.	H. W. R.	131.6	0.60 core.
" 3.	S. W. R.	133.6	No core.
" 3.	"	137.50	No core. (—65.3).
" 3.	H. W. R.	140.40	1.70 core, commenced to core —68.2.
" 3.	"	142.00	0.60 "
" 3.	"	143.50	0.40 "
" 6.	"	145.50	0.25 "
" 6.	"	147.50	0.60 " Lost part flow 147.8.
" 6.	"	148.90	0.75 "
" 6.	"	150.15	0.60 " Lost all flow 149.0.

" M. TIERNEY."

*S. & S. —Sand and stones.
F. S. & S.—Fine sand and stones.
C. S. & S.—Coarse sand and stones.
S. G. & S.—Coarse gravel and stones.
S. W. R. —Soft white rock.
H. W. R. —Hard white rock.

" June 2d. Put in diamond bit at 103.0. Commenced to core at 130.50; rock was not soft like mush; could not turn rods down with the tongs, but was not hard enough to core; did not find any seams or soft spots; stood up good; did not fill in. June 3d, no seams, no soft spots, but not hard enough to core. X Rock in Hole 99 was hard enough to stand up but did not core. Did not find any soft mushy seams. Commenced to core —58.3, cored to —59.40, hard did not core until I got to —68.20. Then I picked up some core all the way down, as report will show, lost part flow —75.60. Lost all flow —76.80. W. J Sager."

Figs. 4, 5 and 6 are three sections of the foundation rock on which the main dam is built. The limits of hard and soft rock surface, as indicated by the soundings, as well as the actual surface exposed upon excavation and the actual surface built upon, are shown. These sections are interesting as a comparison between the possible results, as

shown by the diamond-drill work, and the actual results obtained. In a general way, it may be said that the rock was found to be more broken up and traversed by seams, fissures and soft streaks, in all the various conditions exhibited by limestone ledges, than might have been expected from general surface indications in the neighborhood and from the borings themselves. To a certain extent, the same was true of the gneiss, the surface of which was found to be full of slips and seams running in every direction between hard masses, while extensive pockets and seams of disintegrated rock of considerable

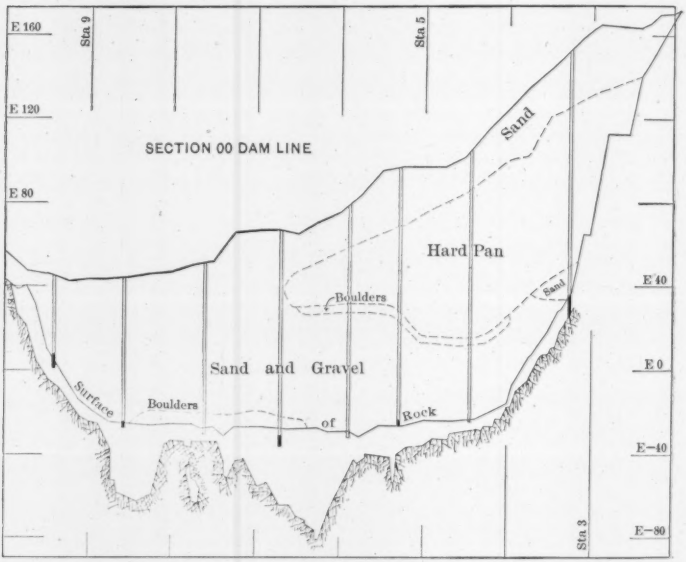


FIG. 4.

width had to be removed or excavated until, in the case of the seams, which were mostly nearly vertical, they narrowed up and nearly or quite pinched out.

The following statement, quoted from the "Report of the Chief Engineer" Mr. Fteley to the Aqueduct Commissioners, 1887 to 1895, is given here in explanation of the fact that it was finally decided to build the dam at this point, although at the time the decision was made all facts in connection with this location had not been devel-

oped, and its superiority to other sites was still an open question, while the additional borings, made, as previously noted, after the site had been decided upon, showed no more encouraging results at least than those made earlier.

"No very favorable location was found, and the writer reported to the Aqueduct Commission on October 8th, 1890, that it would be advisable to abandon for the present the Quaker Bridge site, and to build a dam of less magnitude a short distance below the present Croton Dam (see Location 2, Line C, on Sheets 27 and 29). Although the reservoir to be thus formed would have contained an available

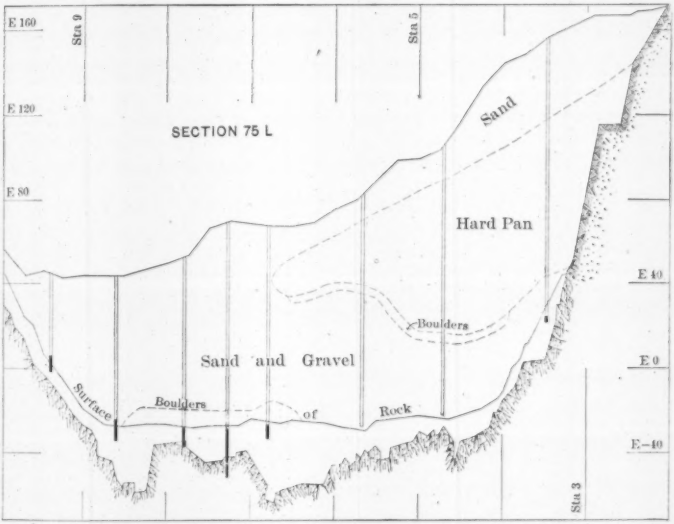


FIG. 5.

storage of about one-half that of the Quaker Bridge Reservoir, the principal reasons given in favor of that opinion were:

"*First.*—That the storage to be thus obtained would be sufficient for many years to come.

"*Second.*—That the height and cost of that dam would be much less, and that it could be built in less time.

"*Third.*—That the experience which would soon be acquired of the effect of the large storage reservoirs under construction on the quality of the water, would better enable the authorities in charge to ascertain whether it would be of good policy in the future to build the higher dam or to resort to some other mode of increasing the supply.

"*Fourth.*—That the interest of the money thus saved for the present would, after twenty-five years, represent a large part of the money necessary to then build the higher dam, with the result that the city would then have two dams instead of one for nearly the same expenditure.

"The report also mentioned that another site (the Cornell's site), not then fully explored, presented good features and should be further considered.

"The Aqueduct Commissioners voted to adopt the last-mentioned site, which is one mile and a quarter above Quaker Bridge.

"Borings made subsequently to this decision disclosed that the rock strata, at places, were found to be at a greater depth than was anticipated; hence, the excavation will be deeper than was originally intended, and the bulk of masonry will be correspondingly larger."

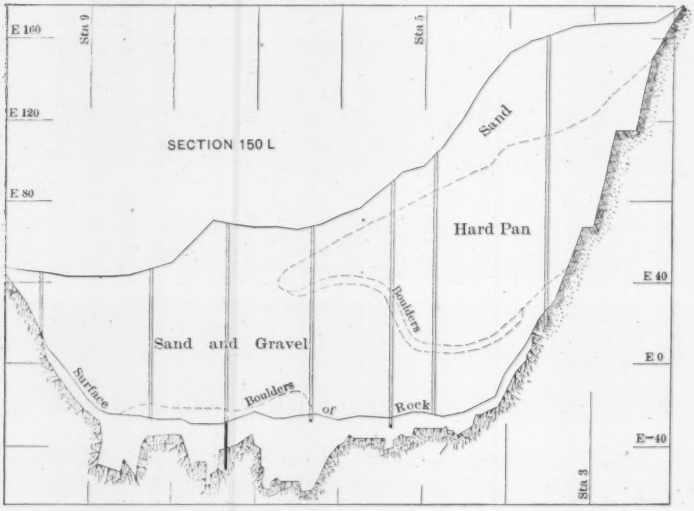
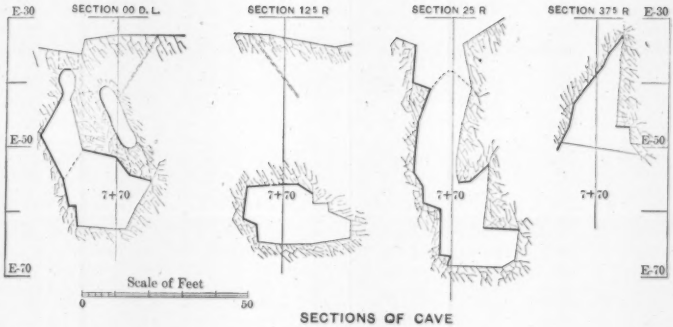


FIG. 6.

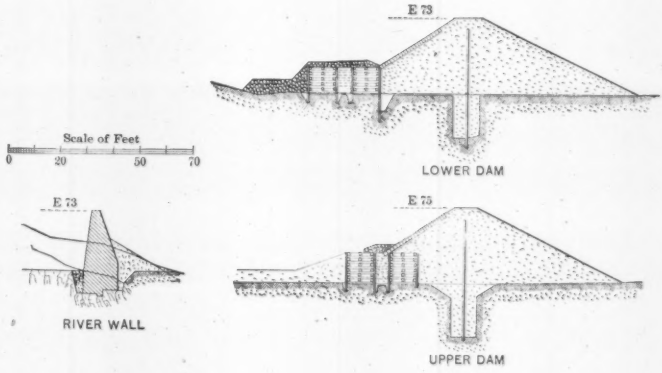
PROTECTIVE WORK.

Plate XXXV shows the general plan of the protective work designed and built for the purpose of enabling the deep excavation necessary for the main dam foundations to be carried on with the smallest chance of interruption from floods.

The section of the river wall which separates the new river channel from the main excavation is shown by Fig. 7. This wall is 600 ft.



SECTIONS OF CAVE



SECTIONS OF WING DAMS

FIG. 7.

long, and is built on the underlying gneiss, care being taken, throughout its length, to make its bond with the foundation rock as complete as possible, particularly throughout that portion which comes within and forms a part of the main dam. Throughout this section, excavation was made in the foundation rock to a considerable depth to get below open seams and fissures, and during its construction a considerable portion of the foundation of the main dam overlying the channel cut was laid up to the grade of the channel, advantage being taken of the necessary foundation work of the adjacent river wall to do it.

Fig. 7 shows also sections of the upper and lower earth wing-dams as built; and their position, relative to the main excavation cut, is shown in Plate XXXV. The main lines of 3-in. sheeting, which were relied upon as the water stops, were carried down from 20 to 25 ft. below the original surface. As most of the material in which this sheeting was placed was coarse, loose gravel and sand, resort was had to trenching, with sides temporarily sheeted, and the permanent sheeting, after being placed in position, was driven by heavy hand mallets down an additional foot or two. At the east end of the upper wing-dam, however, for a considerable distance, the bottom was found to be of quicksand, and the sheeting was put down, through a considerable part of the depth reached, by means of a water-jet and heavy hand mallet.

The crib-work is designed to protect the embankment toes from the great erosion to be expected in case of a heavy freshet, while the extra sheeting and loading of stone on the lower wing-dam crib is a still further protection against the wash of the discharging channel, which, in extreme cases, might be strong enough to displace the loading and possibly cause a slight movement of the cribs which, in such cases, are so planned as to yield measurably outward without materially endangering the toe of the embankment.

While the river channel and these dams are designed to carry in emergency 22 ft. of water, or more, it may be said that at the present writing the deepest flow experienced through the channel has been about 11 ft. This was due to a warm rain of 3.6 ins., most of which fell in about 12 hours, on 3 ins. of snow lying on deeply frozen ground, in the month of February. From this it is easy to see that a combination of circumstances resulting in a flow which would tax the channel to its full capacity is quite possible.

THE AQUEDUCT COMMISSIONERS

CONTOUR PLAN OF NEW CROTON DAM

Jan. 1899

A. FTELEY
CHIEF ENGINEER

CHAS. S. GOWEN
DIVISION ENGINEER

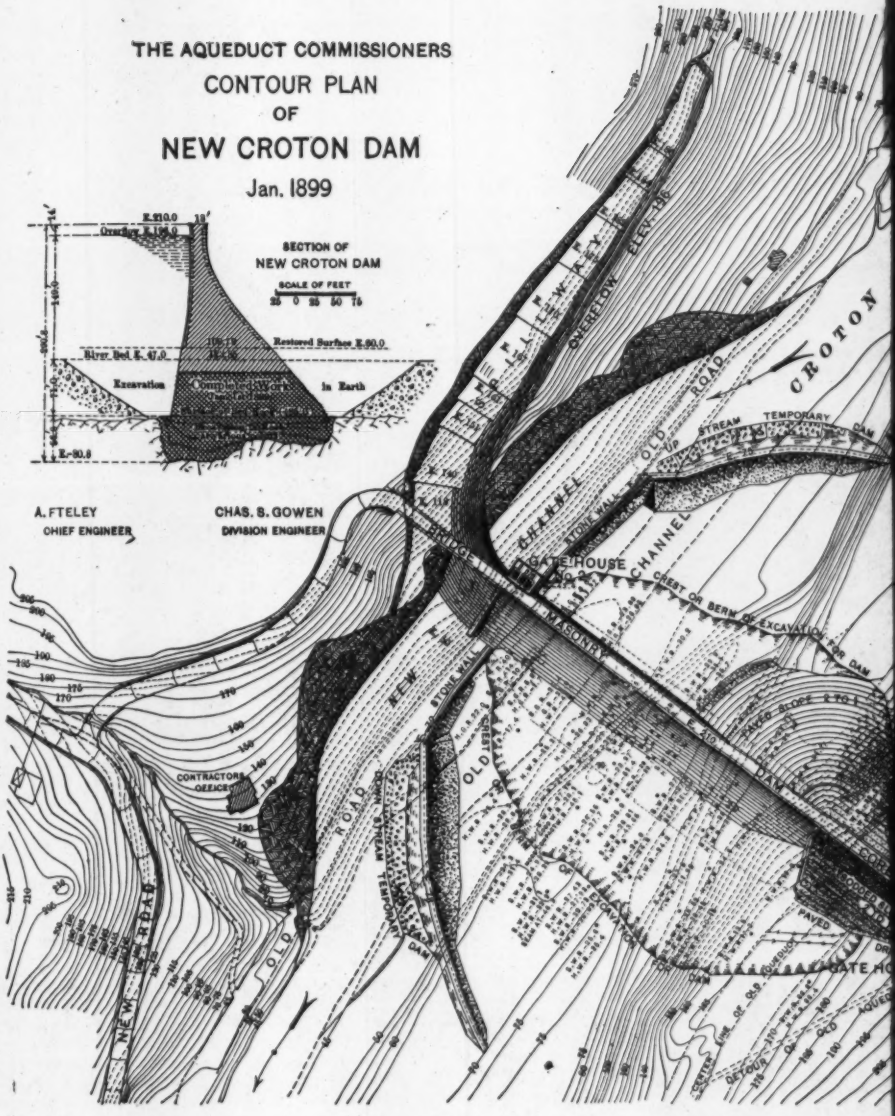


PLATE XXXV.
TRANS. AM. SOC. CIV. ENGRS.
VOL. XLIII, No. 875.
GOWEN ON FOUNDATIONS OF NEW CROTON DAM.

Before leaving the subject of the protective work, attention is called to the somewhat extensive and perhaps seemingly permanent character of its design and construction. This work involved, in the construction of the river channel, an earth excavation of about 100 000 cu. yds., and rock excavation of about 106 000 cu. yds. The river wall and wing-dams include in their construction:

Earth Excavation	8 500 cu. yds.
Vertical Trench Excavation	6 700 " "
Refilling and Embankment	58 000 " "
Rock Excavation	4 000 " "
Timber	390 000 ft. B. M.
Crib Work	7 000 cu. yds.
Rubble Masonry	10 000 " "
Paving and Rip Rap	2 000 " "

While the cost of the above work is a large amount (upwards of $350 000), its proportion to the total cost of the dam, which may amount to $5 000 000, is not excessive, and it must be remarked that a considerable portion of it will form a part of the permanent structure. It seems to have been justified on account of the very efficient protection it has afforded to the extensive excavation work, both of earth and rock, and the foundation masonry work, which have been carried along steadily for three years and which, in the case of the masonry and refilling, must continue for another year at least before the dams will cease to be necessary. The extreme depth of the pit, in which the work has been done, below the river bed, is 130 ft.

Earth Excavation, Main Dam Foundation.

This work involved preparation for a foundation on rock extending from about Station 3 + 30 to about Station 10 + 00, where the new river channel, formed in connection with the protective work, is merged into the foundation, and which varies in width from about 200 ft., at the lowest point, to about 130 ft. at Station 10 + 00 and 140 ft. at Station 3 + 30, on the line of the back of the proposed wing-wall (see Fig. 3). The necessary earth excavation covering this area was about 885 000 cu. yds., consisting largely of loose sand, gravel and boulders with, however, at the south end of the pit, a large area of hardpan excavation, this hardpan forming, to a considerable extent,

the slopes at this end of the excavation, and extending in depth at the extreme south end, *i. e.*, the point of junction of the main dam masonry section with the core-wall, from the surface of the bed rock to about Elevation 130, above which it was surmounted by loose, fine sand reaching to the surface. Figs. 4, 5 and 6 are sections indicating at various points the relative positions of the different kinds of earthy material which had to be moved, and the south end slope lines to which the excavation was made. In the case of the gravel and sand, the slopes were $1\frac{1}{2}$ horizontal to 1 vertical, and in the hardpan at the extreme south end $\frac{1}{2}$ horizontal to 1 vertical, with a berm about halfway up the slope; while on the quarters, where the depth was considerably greater, the slopes and berms were varied somewhat, as the end slopes were merged into the side slopes. In laying out the slopes, consideration also had to be given to the fact that on the quarters, at a comparatively low elevation, the hardpan was underlaid with layers of boulders and gravel which extended to the bed rock as it dipped in its surface between Station 3 + 30 and Station 5 + 00, where it reached the general level of the rock in the valley bottom. These earth slopes were all planned to allow for a toe berm of 20 ft. in width, at the rock surface, and this space proved to be necessary and useful in further operations in the rock bottom below.

The slopes stood very satisfactorily, on the whole, no particular trouble resulting from washing or sloughing, in case of the gravel slopes, so long as surface drainage outside the pit was properly controlled. In the case of the hardpan, steep slopes which in combination with the sand above and at certain points with sand, gravel and boulders below, were, at the maximum, 150 ft. in height, the only trouble experienced was during the open winter of 1897–98, when successive freezings and thawings caused the slope surface to slough off in successive thin layers representing in thickness the depth in each case to which the frost had penetrated since the preceding thaw.

The maximum width of the pit, from the top of the up-stream slope to the top of the down-stream slope, was about 600 ft., and the largest area of cross-section excavation, above bed rock and parallel to the trend of the valley, *i. e.*, at right angles to the dam line, was about 49 000 sq. ft.

Figs. 3, 4, 5 and 6 show in plan and section the crest and toe lines of the slopes and the location and elevation of the berms in the steep

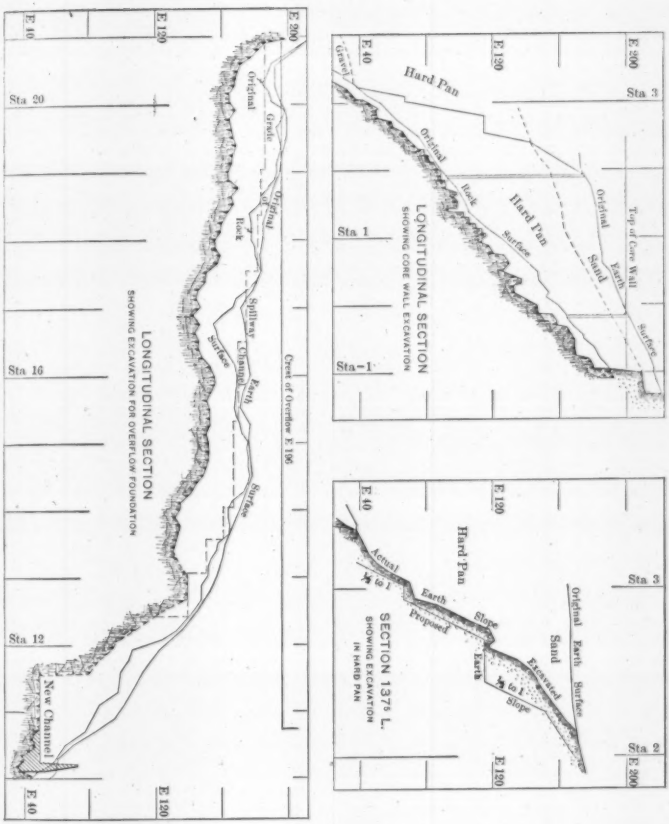

LONGITUDINAL SECTION
SHOWING EXCAVATION FOR OVERFLOW FOUNDATION
Sta 20
Sta 16
Sta 12
E 200
E 130
E 40
Original Grade
Rock Surface
Spillway Channel
Pond
Crest of Overflow E 196
New Channel
LONGITUDINAL SECTION
SHOWING CORE WALL EXCAVATION
Sta 3
Sta 1
Sta -1
Gravel
Hard Pan
Original Rock Surface
Top of Core Wall
Sand
Earth
Surface
SECTION 137½ L.
SHOWING EXCAVATION IN HARD PAN
Sta 3
Sta 2
Original Earth Surface
Excavated
Actual ¼ to 1 Slope
Proposed ¼ to 1 Slope

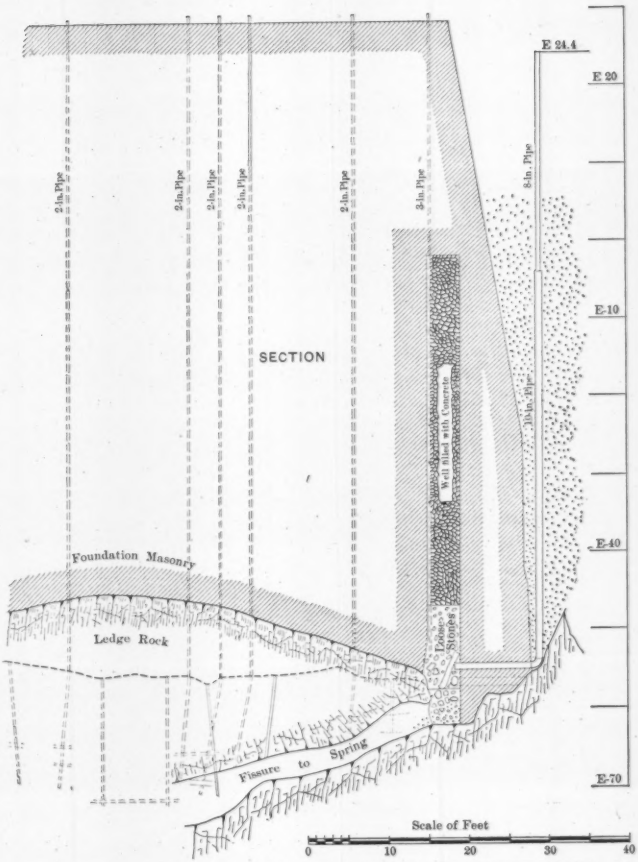
FIG. 8.

slope at the south end of the pit. The line of the masonry foundation is also indicated and its connection with the core-wall. A section at 137.5 L., in Fig. 8, shows the ordered and actually excavated slope in the hardpan at its highest point.

A very large amount of this excavation, lying on the south slope of the valley and above the level of the river, was removed by steam-shovels, three of which were in use at one time. The first work done in sinking below the river bed was by means of a large "orange peel" dredge, specially constructed for the purpose and used for the excavation of the loose gravel and sand until the near approach in depth to bed-rock, and the necessity of beginning rock excavation, demanded a change in methods, as the dredge work was dependent upon a certain depth of water in which to work the bucket efficiently, while the rock excavation rendered close drainage necessary. For the further prosecution of this work resort was had for some time exclusively to three cable-ways stretched across the valley longitudinally along the line of the dam at such transverse intervals as to cover the plan of foundation. These cables were installed for the purpose of aiding the earth and rock excavation and, ultimately, for taking in stone and other material for the dam masonry. They were used for some time in connection with the dredge above mentioned, and were in turn supplemented, when the rock excavation work assumed large proportions and there was considerable earth work remaining, by railway inclines placed successively at different points on the side slopes and worked by means of stationary hoisting engines and cables.

With the use of railway inclines, steam-shovels were again operated, and a large amount of coarse indurated gravel, lying just above the bed rock at the north end of the main cut, was thus excavated, and, as the excavation progressed toward the south end of the cut and the hardpan was reached, it was removed almost wholly with the aid of heavy steam-shovels, although the slope trimming at the south end and on the quarters, and some bottom cleaning up on the rock surface, had to be done by hand with the aid of skips and derricks.

Fig. 1, Plate XXXVI, shows a part of the river wall and lower wing-dam, and the progress of the main dam excavation to September, 1895. The large pit shown was excavated mainly by means of the dredge, shown on the extreme right, with considerable assistance from the cableways, for which the material was excavated by hand into large



FISSURE AND SPRING AT STA 5+95

Piped holes shown thus •

FIG. 9.

scale pans, and hoisted and transferred to the foot of the heavy slope shown in the rear, where it was dumped into cars. The levels above the pit, shown in Fig. 1, Plate XXXVI, were excavated with steam-shovels, and later, as further progress was made into the hard material of the great slope at the south end of the cut, steam-shovels were again placed at a lower level in the pit and the inclines were used as mentioned previously. Fig. 2, Plate XXXVI shows more particularly the steep slopes in the hardpan as finally shaped, and Fig. 1, Plate XLV shows in a more general way the side slopes, but at some time after the excavation was completed, when a small amount of back-filling had been done. Also, at this time, cuts had been made in the side slopes, forming berms on which side tracks were laid, to furnish supplies for the foundation masonry. Fig. 2, Plate XXXVI, shows particularly the hardpan slopes ($\frac{1}{2}$ horizontal to 1 vertical) and berms at the south end and on the quarters, as well as the underlying rock bottom excavated for the foundation masonry.

Core-Wall Excavation.

The core-wall extends from the south end of the main dam for a distance of about 430 ft. into the side hill. Its general section and the cross-section of the trench excavated for it are shown in Fig. 1. The maximum width and height of this wall, which occurs at its junction with the main dam masonry, are, respectively, 18 ft. and 175 ft. The material excavated for the wall was hardpan above the limestone foundation up to within a depth from the original surface varying from 24 ft. to 8 ft. Above this hardpan were gravel and sand. The general extent, as well as depth of excavation for this wall, together with the line limiting the top of the trench excavation, are shown on the profile, Fig. 8.

The trench walls were vertical, the sustaining power of the hardpan allowing the sheeting and bracing to be done after the completion of the successive levels excavated, which levels varied from 6 to 12 ft. in height or depth, according to the depth of the section of trench then under excavation. As stated, the hardpan, throughout the length of this trench, extended to the rock foundation, which showed considerable variation in hardness and texture, and called for excavation of considerable depths below the rock surface in certain places before compact layers of sufficient hardness were found. Fig. 1, Plate

FIG. 1.—FEBRUARY 7TH, 1896. VIEW FROM UPPER END OF FOOT-BRIDGE, LOOKING S. E. FLOOD IN FOREGROUND.

FIG. 2.—OCTOBER 1ST, 1897. SLOPE AND BERM AT SOUTH END OF CUT.

XXXVII, shows the rock bottom ready for the masonry of the core-wall, as well as the sheeting and sides of the trench for a certain distance up, at Station 1 + 80, 150 ft. from its junction with the main dam. At this point the rock was sufficiently compact and of necessary bearing strength, although not very hard, and the steps shown in the inclined surface of this foundation were made with picks and shovels. The depth to which the rock was excavated varied from 4 to 7 ft.

The width of the trench is measurably greater than the thickness of the core-wall, and the difference was liberally planned in order that there should be no difficulty in finding working room at the bottom of the trench to remove the bracing and sheeting after the masonry foundations of the wall were started. It also gave proper opportunity to place the refilling; which was of the same material as had been excavated, and was placed very carefully in layers varying from 2 to 4 ins. in thickness, and thoroughly rammed by hand. Advantage was also taken of this extra width to widen the footing or lower courses of the core-wall, thus increasing the bearing surface in certain places where the rock foundation might possibly call for it, and the section shown in Fig. 1 is taken at one of these points.

As to the thickness of this wall, which it will be noted is somewhat massive, varying from 6 ft. in thickness at the top to 18 ft. at the lowest point, it may be said that the wall was purposely designed not only as a water-tight screen reaching from bottom to surface between the upper and lower sections of the enclosing embankment, but also to afford a substantial resistance to any overturning or crack-producing force which might be caused in the course of time by the saturation of the up-stream bank and its consequent increase of unit weight.

The maximum depth of sheeted vertical trench excavation, including the depth of excavation in the foundation rock, was 75 ft. This point was at Station 2 + 50. At this point the top of the vertical trench was 27 ft. below the original surface of the ground. The earth material above the core-wall trench level was excavated by steam-shovel; below, in the trench proper, it was excavated by pick and shovel, and removed by derricks. Black powder was generally used in sinking the trench, at the lower levels particularly, to loosen the hard-pan, and it was used very extensively for the same purpose in the main cut, both for facilitating the work of the steam-shovels and for all handwork done in the removal of hardpan.

Overflow Excavation.

At the present writing, the completed overflow foundations, embracing a length of 750 ft., extend along the side hill on the north side of the river and finally abut in the rock of the hill at the upper end.

This rock foundation is entirely country rock, or gneiss, and the amount of superimposed earth was not large, and was mostly sandy loam on the surface, with underlying gravel.

Fig. 8 shows a profile of the earth and rock excavation as well, and on Fig. 1 are shown representative cross-sections, indicating more clearly the extent to which rock excavation was found necessary to insure a fairly tight bottom. The rock was full of seams and faults, and considerable depths had to be reached at certain points in order that open seams running across the line of the structure might be followed until they pinched out. The extensive rock excavation in the front of this foundation work, shown in the cross-sections, was necessary to provide the waterway leading from the spillway bottom to the old river-bed below the main dam.

Rock Excavation and Foundation for the Main Dam.

As stated in the general description of the dam, the rock on the north side of the valley, on the steep side hill, cropped out at points very near the surface. It was formed of gneiss, considerably fissured, but generally sound after reaching a certain depth in the ledge. This gneiss extended to the line of the old bed of the river, where its depth below the surface was much greater, being about 75 ft. The section under consideration was found to be well broken up near the surface by open seams of considerable width, varying from 2 to 3 ins. in cases. Such seams were filled with earth, and extended in all directions. There were also some strata of rock, more or less disintegrated. These varied from 1 to 3 ft. in width or thickness, and were removable with pick and shovel for some depth from the surface. The dip and strike of this rock were about the same as that of the limestone beyond; the dip being nearly vertical and the strike following the line of the valley at right angles to the dam.

The profiles, Figs. 4, 5 and 6, show the excavation necessary to get below the loose and open seams, which in places was considerable, as

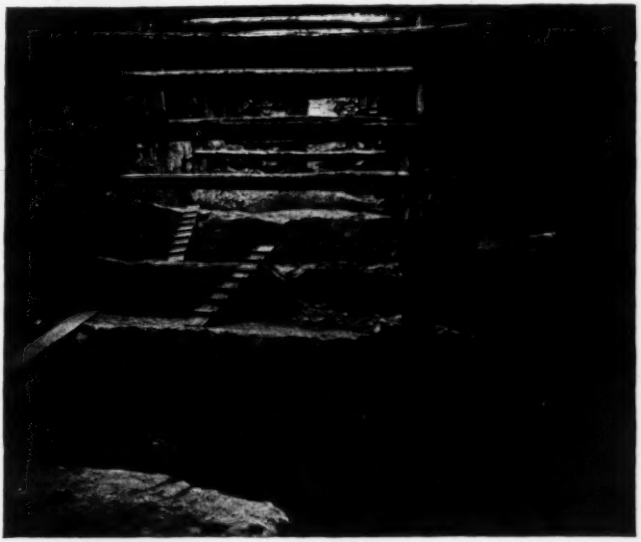


FIG. 1.—MAY 27TH, 1897. STEPPING AT BOTTOM OF CORE WALL.

FIG. 2.—MAY 26TH, 1896. LAYING FIRST STONE IN DEEP ROCK CUT.

the seams separated the rock into heavy solid masses which they bounded on all sides. The bottom reached was solid, compact and tight. At the date of writing, a narrow strip of this bottom, close to the river wall foundation, remains to be excavated. This was left, when the great bulk of the work was completed, as, at that time, it furnished the foundation for a trestle then in use. This remaining strip is about 20 ft. wide, and, with the section of the overflow bottom now under process of work, completes all that is incomplete in the foundation excavation of the main dam.

Under and beyond the river-bed, the character of the rock changes entirely, being composed wholly of limestone. The two rocks were separated by a well-defined, nearly vertical layer of shale, black in color, especially on the up-stream side, friable on the surface, but becoming harder a few feet below, particularly on the down-stream half of the foundation. The welding of the two main rocks, the gneiss and the limestone, with the shale, appeared to be quite complete at the depth of excavation finally reached. The surface of the limestone, from the point of junction toward the south, was nearly level for a distance of about 400 ft., until it reached well into the other side of the valley, where it rose gradually with the south slope. The limestone varied greatly in character throughout the extent uncovered. In places it was of sufficient compactness and water-tightness to answer for the foundations of the structure. In other places the general character was diversified by belts of varying width which were either full of eroded seams, through which water was found to flow freely when excavation was in progress, or masses of stone broken up by seams running in all directions, which were filled with mud. In addition, there were other well-defined belts, and all followed the general dip and strike of the rock, which, in the case of the dip, was nearly vertical, and of the strike, at right angles to the line of the dam, following the valley. These last belts were of partly disintegrated, finely granulated limestone; were very well defined and at the surface were easily removable with the pick; and grew harder and more compact with increased depth of excavation. These fissured, eroded and granular belts seem to form three distinct classes into which the bad features of this limestone bottom may be separated.

The different fissures developed many erosions in certain cases and were found at various points through the limestone stretch of the

foundation, being larger and closer together as the junction with the gneiss was approached. These fissures, while well defined, were of varying widths, developing a line of erosions generally through very hard limestone.

As an illustration of the eroded seam, one case developed into a cave, the location and existence of which were noted by tracing a narrow, horizontal seam in the rock near the surface, at about Station 7 + 70, 50 L., along the strike of the rock. This seam was in fairly solid rock, and clear water flowed from it. As the excavation along the line of this flow toward the up-stream side of the dam progressed, there was found a sharp downward dip, and the flowing stream soon required for its management a subsidary pump. The seam enlarged into an erosion filled with sand, which, as it was followed, developed into a cave about 7 ft. x 9 ft. in section. This led under a heavy mass of solid rock to and beyond the up-stream line of the dam foundation. Connected with it were found, on the sides and in the roof, other erosions which were traceable nearly to the surface of the rock within the limits of the dam foundation, and which, on the up-stream side, outside face limits, in one case penetrated to the rock surface, where it showed as a narrow and somewhat prolonged fissure.*

While all the eroded fissures showed flows of water of varying degrees, several such were found which developed into strong springs, of which special care had to be taken. One, in particular, was found as the excavation in the rock deepened, limited and defined to an erosion in solid rock 6 or 7 ft. in diameter at about Station 6, near the up-stream side. The flow here was continuous and heavy, more than filling the 10-in. pipe which was at first placed to receive it, and afterward, as the spring hole was welled up in the foundation masonry, rising with this masonry and in the pipes which were at the same time placed in connection with the well, to a height of 90 ft. above its source before it was found advisable and expedient to attempt to fill it up and block it off. A particular and detailed account of all the operations connected with this spring will be found in the particular description of the treatment of the rock bottom.

As to the granular belts referred to, the excavation in them was carried down until the surface exposed was very compact. These surfaces were afterward tested for bearing power by means of an arrange-

* A detailed description of this cave will be found further on.

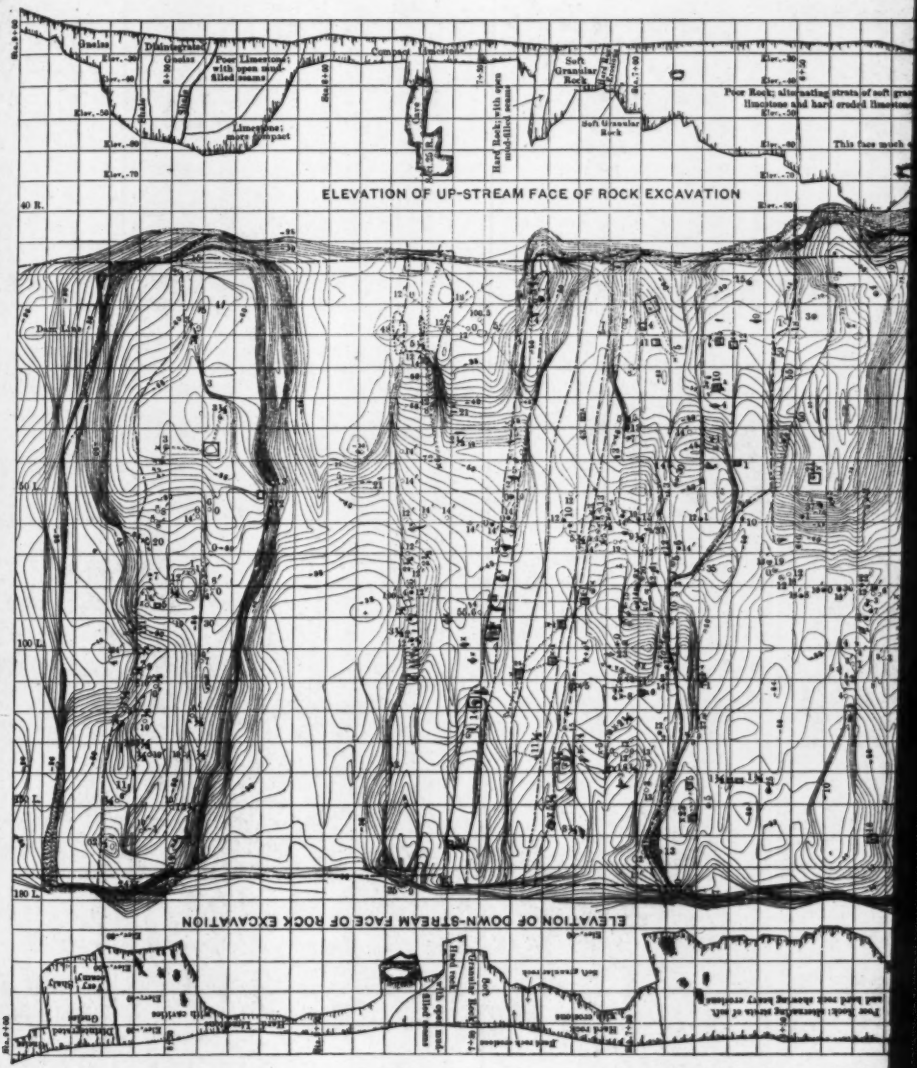
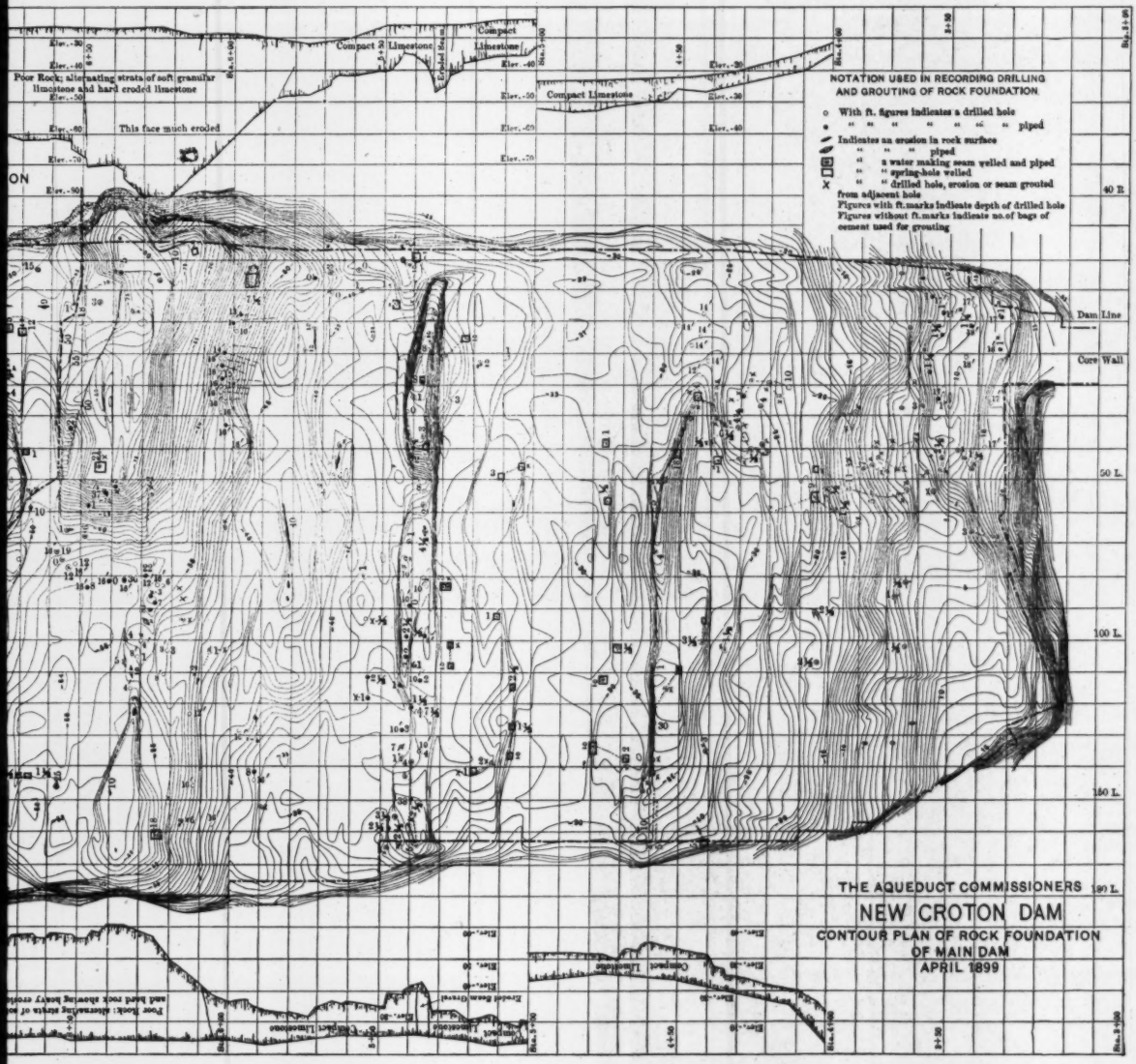

ELEVATION OF UP-STREAM FACE OF ROCK EXCAVATION
ELEVATION OF DOWN-STREAM FACE OF ROCK EXCAVATION
Dam Line
40 R.
50 L.
100 L.
180 L.

PLATE XXXVIII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. XLIII, No. 875.
GOWEN ON FOUNDATIONS OF NEW CROTON DAM.



ment especially designed for that purpose and shown in Fig. 10. Further allusions to these belts will also be found later.

In limiting the extent of the excavation vertically, the end aimed at was to reach rock sufficiently free from seams, and solid enough to afford all the bearing strength necessary to sustain the superimposed masonry and resulting pressure. The result involved a very large amount of deep rock excavation; the depth in one place being 50 ft. before satisfactory compact rock was found. It is not assumed that there may not be some tendency to upward pressure through some of the fissures which remained after the excavation was completed, but, as will be described later, every effort was made and every precaution taken to fill them, and it may be conceded that should upward pressure occur in some cases it must be reduced to the very small area presented by the mouth of the fissure in question to the bottom layer of the dam masonry, this area forming a very small proportion of the greater area against which upward pressure might be expected.

As to the possibility of percolation under the dam, that question would be more important if the rock bottom were exposed to the direct contact of the water in the reservoir, but it must be borne in mind that from the lowest point of the foundation of the main dam to the top of the back-filling above, there will be a compact filling of about 150 ft., in this particular case, which, while extreme, is not different, excepting in the great depth, from the condition which will obtain along the whole length of the masonry dam.

This question of possible percolation will be further considered in connection with the chapter on "Pumping."

It is an important and peculiar fact that, throughout the rock excavation of the whole foundation, in no case did the numerous test holes, drilled in the vicinity of seams and erosions, strike any openings in seams or rock which were not easily traceable by some continuous natural passage to the surface of the rock under preparation for the foundation. In other words, it may be fairly claimed that the existence of all open seams lying within 12 to 16 ft. of the dam in the various bad sections was traceable from natural indications at the surface. It is, therefore, to be assumed that all such seams were found and properly noted. A reference to the contour plan, Plate XXXVIII, will show that the variations and character of the limestone, and the necessary excavation, were much greater nearer its junction with the

gneiss than at the south end, where, with the exception of a few eroded seams, the rock is uniformly hard and compact, and required but comparatively little excavation at the surface. It would seem that at some time the disturbance of the limestone formation must have been considerable; the greater part of it occurring near the point of junction. From developments indicated by a comparatively small amount of excavation in this part of the limestone foundation, and the fact that the general character of this bottom was naturally considered an important matter, it was deemed advisable, during the excavation of the first section of the bad rock, which lay at Station 8 + 50, to consult a specialist as to the general condition in which limestone ledges might be expected to be found under the prevailing conditions, and Professor Kemp, of Columbia College, was consulted, and his attention was called particularly to the question of the probable existence of caves and similar openings under the general rock surface. The following is his report, which is introduced here as being of interest under the circumstances :

"NEW YORK, May 14th, 1896.

"Mr. A. FTELEY,

"*Chief Engineer, Aqueduct Commission.*

"MY DEAR SIR,—In reply to your letter of the 12th, requesting me to report also upon the probable condition of the limestone under the site of the dam, I append the following to my report of two days ago.

"The limestone is undoubtedly more or less fissured precisely as is the gneiss and as is to be expected in regions where the rocks have been upturned to a vertical position.

"Such small cracks cannot of course be avoided and, I understand, are not matters of serious concern. They are the ones that now show in the walls of the pit and that let in the water in all probability from the overflowing water-soaked sands and gravels.

"As to the presence of large caves, several feet across or more, and of great length, I am of the opinion that their existence is improbable, and so improbable as not to give occasion for special treatment. I think the points *For* and *Against* them may be stated as follows:

"FOR.

"1. The rock is limestone, and caves are practically limited to limestone; other soluble rocks being rare with us.

"2. Mr. Value has been impressed with the fact that the water trickling into the sump has diminished as the pit has gone deeper. The inference has been suggested from this observation that the water has run away into some underground cavity. (See further, under 5, below.)

"A third point is stated and discussed under 2, below.

FIG. 1.—MAY 26TH, 1896. MAIN DAM EXCAVATION. LOOKING SOUTHWEST.

FIG. 2.—MAY 19TH, 1896. MAIN DAM EXCAVATION. DEEP ROCK CUT, LOOKING WEST.

"AGAINST.

"1. Caves only form above the level of the ground-water or well-water, because only freshly fallen rain is sufficiently charged with carbonic acid to be a strong enough solvent to be serious, and because only water in this situation flows rapidly enough to produce profound effects. The ground-water stands too still, and too soon becomes saturated with lime, to be effective. The present position of the rocks is below the zone at which caves could form, and it is practically assured that none have formed since they assumed this position.

"2. If any have formed, they must have done so when the rocks stood at a higher position and above the ground-water. We all believe that this whole region was much elevated during the Glacial period, and it cannot be denied that conditions may have been favorable at that time. Some superficial decay apparently took place, as shown by the sandy streaks in the limestone, but after this time a strong stream must have flowed over these rocks to have availed to deposit the heavy burden of sands and gravel that rest upon them, and if any such cavity existed near the surface the probability is strong that it has been packed full of sand.

"3. The rocks stand vertically, and all underground drainage or circulation must tend to follow their bedding planes much more than to cross them. We would infer from this that any cavity would be long and narrow and not an easy thing to locate with a drill.

"4. No hollow sound, so far as I know, has been noted in the work in the pit, when picks, drills or the descending boxes from the cables have struck the bed-rock.

"5. In case the water has diminished, as observed by Mr. Value, I think it is due to the partial exhaustion of the neighboring gravels, for the weather has been dry and rainless for a long period, rather than to any cavities under the bottom of the pit. Such assumed cavities, being 50 to 100 ft. below the level of the Hudson River, and having stood for an indefinitely long time under wet gravels, must have been long since filled with water.

"6. All the experience, so far as I know, that has been gained in quarries in these limestone belts in New York and the neighboring parts of New England, has shown caves to be extremely rare. An assistant of mine has recently had occasion to visit every one of them, and he only met one small cave, which was at Hastings. Of course there may be others, and I am aware of the existence of a large cave near the Twin Lakes in the northwest corner of Connecticut, but, considering the abundance of the limestone areas, they are certainly rare.

"For these reasons I regard the probability of their existence under the site of the dam as remote.

"Very respectfully yours,

"(Signed) J. F. KEMP."

In order to explore this limestone bottom more completely, it was found advisable to drill a few test holes of considerable depth. These were accordingly undertaken at certain points in the bottom where it was nearly ready for the masonry, and their location, direction and depth, are shown on the contour plan.

An extended account and description of the excavated rock bottom, for the main dam foundation, referring particularly to the limestone bottom and to the various changes and characteristics shown by this rock, can be found in a report made by the author to the Chief Engineer of the Aqueduct Commission, in which, for the purpose of a record, all the facts are noted in considerable detail. Constant reference is had to Plate XXXVIII, which is a plan in contour of the finished rock bottom between Station 3 + 20 and Station 9, and which shows contour elevations at intervals of 1 ft. On it are shown also, by means of cross lines, the limits of the different belts in the rock, and the heavy dotted line shows the neat lines of the dam foundation masonry.

The shale seam, separating the gneiss and limestone, lies at Station 8 + 80 ±, varied in width at different points, but grew narrower and more solid as the depth of excavation increased. Fig. 2, Plate XXXVII, shows the shale in the face of the cut and in the trench bottom on the left, where, however, the excavation shown is unfinished. The view is taken looking up stream.

The next of the series of belts into which the foundation may be divided, and which are in a measure indicated by the profiles of the finished bottom shown in Plate XXXVIII, and in Figs. 4, 5 and 6, extends from Station 8 + 20 to Station 8 + 70. Its character, when the excavation was about completed, is shown in Fig. 2, Plate XXXVII, and Figs. 1 and 2, Plate XXXIX. The contour plan, Plate XXXVIII, shows the number and depth of the search holes drilled in preparing the bottom for masonry. In this case they followed principally the lines of the erosions in the hard rock bottom.

Next beyond lies a section, showing also in Fig. 1, Plate XXXIX, between Stations 7 + 80 and 8 + 20, and forming a solid ridge of hard, compact limestone, requiring but little excavation, comparatively.

A well-defined narrow seam along Station 7 + 70 is illustrated by Fig. 1, Plate XL, showing its down-stream end. Its up-stream end

FIG. 1.—SEPTEMBER 1ST, 1896. MAIN DAM EXCAVATION, LOOKING WEST.

FIG. 2.—SEPTEMBER 12TH, 1896. CAVE AT STATION 7 + 70, 10 L.

developed upon excavation into the cave previously referred to. Fig. 2, Plate XL, shows the cave opening at Station 7 + 70, 25 L. The pump and suction hose in use are also shown. This suction hose and another line, of the same size, were built in the masonry when the tunnel was filled up, in order that the necessary drainage from a sump hole, placed outside of the upper line of the dam in the lowest point of the tunnel reached, might be maintained. The length of the cave excavated and filled as tunnel was about 30 ft. The floor is of very hard and solid rock; holes traced 16 ft. deep found no openings below. The masses of rock on the top and sides are all solid, showing few or no open seams, except that the seam between the cave floor and the right side wall may have had an open connection with the low point in the excavation at Station 7 + 35, 15 R., where, in grouting, later, there were some indications of an open passage between the points in question. As this was indicated by the pump from the sump at Station 7 + 62, 25 R., throwing out grout which was being pumped in at the point noted, it is not at all certain that the line of communication between the two did not lie mostly outside the dam foundation.

To facilitate the work of filling up the cave, a small shaft was sunk at Station 7 + 73, 23 R., to strike one of the subsidiary caves found on the upper line of the dam. Two other and smaller eroded chambers, leading into the roof of the main cave, were also found. These spaces were all filled, within the lines of the dam, with rubble masonry, or, in the case of the two small caves shown in plan on the contour plan at Station 7 + 70, and Station 7 + 78, with well-packed small stone, placed from above through openings made for the purpose in the overhead rock, and then filled with grout.

The sections and sketches opposite Station 7 + 70, Fig. 7, show the location and extent of these various caves, and it may be noted that in filling up the main cave and the branch cave, *i. e.*, entering above the main cave roof from above the up-stream line of the foundation masonry, care was taken to build this masonry filling 4 ft. beyond the limiting line. The large cave beyond the line was found to be about half full of sand and gravel when it was reached from the tunnel and the small shaft sunk in its roof. It is evident that originally the space had been solidly filled, but that the cleaning of the tunnel, and the pumping of the heavy water flow had caused the partial emptying of the filled space beyond. It was refilled later, after the masonry

filling had been built, by washing gravel down from the slope outside through openings made in the surface of the rock for that purpose.

Between Stations 7 + 30 and 7 + 60 is shown a narrow well-defined seam of hard rock with many erosions connected and extending to the deeper holes excavated at the ends. Fig. 1, Plate XL, shows on the left the deep excavation on the down-stream end. Beyond this seam lies a compact seam of friable limestone about 10 ft. wide. It is shown in Fig. 1, Plate XL, on the extreme left, where the ladder is resting against it, when the excavation was completed. It was tested for bearing strength by an apparatus shown in Fig. 10. This apparatus consists of a cylinder to be loaded with shot necessary to produce the required pressure upon its bearing point, a circle ¼ in. in diameter. This was applied carefully and repeatedly to the surface in question at different points, and the results indicated that the bearing power of the surface was ample up to the limit of the test, which was 250 lbs. to the square inch.

A second but much narrower section of the soft granular rock just mentioned lies a short distance beyond. This shows on Plate XL, as from 3 to 5 ft. in width, and beyond this is a wide stretch of bottom shown on the plan as being composed of alternate strata of "soft and granular limestone and hard eroded limestone." In this the most extensive and deepest excavation occurred. Figs. 1 and 2, Plate XLI, which are views looking up stream, illustrate the character of the excavation. Fig. 2, Plate XLI, shows the deepest point reached, *i. e.*, Elevation—80.4 below datum. Figs. 1 and 2, Plate XLII, show the character of the rock in the same vicinity more in detail, and the location of some of the erosions through which excavation was made. In Fig. 2, Plate XLI, is shown the spring hole at Station 5 + 93, with the iron pipe in use to convey the flow.

The sloping bed of rock shown in Fig. 2, Plate XLI, extends quite across the dam; and beyond it, to Station 5 + 40, lies a hard bottom, reasonably free from seams and erosions, and calling for but little excavation. Between Stations 5 + 30 and 5 + 40 occurs a deep, well-defined seam, or series of erosions, shown at the upper end in Fig. 1, Plate XLIII. This photograph shows also the general character of the rock bottom on both sides of this seam, and Fig. 2, Plate XLIII, and Fig. 1, Plate XLIV, show it still further to the south, and to the junction of the main dam with the core-wall. This part of the bottom

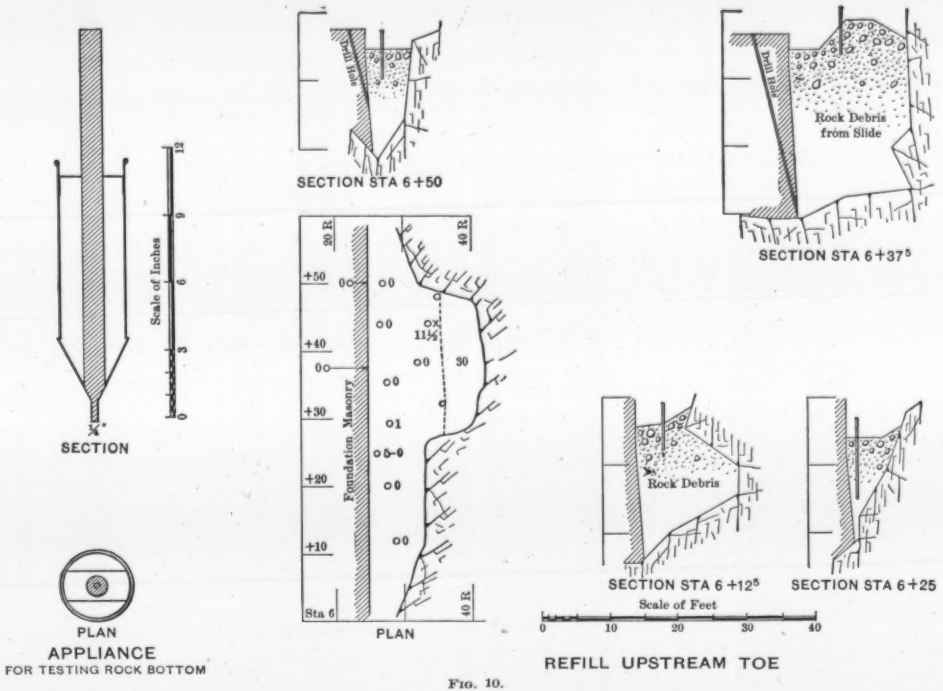

Scale of Inches
0
3
6
9
12
SECTION
PLAN
APPLIANCE
FOR TESTING ROCK BOTTOM
Earth Fill
SECTION STA 6+50
20 R
40 R
+50
+40
+30
+20
+10
Foundation Masonry
Sta. 6
40 R
PLAN
Earth Fill
Rock Debris from Slide
SECTION STA 6+37.5
Rock Debris
SECTION STA 6+12.5
SECTION STA 6+25
Scale of Feet
0
10
20
30
40
REFILL UPSTREAM TOE

FIG. 10.

calls for little comment, although a seam showing at Station 4 + 60, and extending partly across the foundation, is shown nearly ready for masonry in Fig. 1, Plate XLIV.

In addition to the search holes, which were numerous, and were from 12 to 14 ft. in depth, it was thought advisable to drill a few test holes of considerable depth. They were accordingly undertaken at certain points in the bottom where it was nearly ready for masonry, and their location, direction and depth are shown on the contour plan. The first, or No. 1, was located at Station 8 + 68, 103 L. Length drilled, 48.4 ft.; direction and dip indicated by the black arrow ending at 8 + 52, 100 L. Hole No. 2 was located at Station 7 + 76, 84 L. Length drilled, 100.6 ft.; direction and depth indicated by arrow ending at Station 7 + 16, 86 L. Hole No. 3 located at Station 7 + 52, 5 R. Length drilled, 100.6 ft.; direction and dip indicated by arrow ending at Station 7 + 14, 50 L. Hole No. 4 is located at Station 7 + 50.5, 75.8 L., and was drilled vertically 55.6 ft. deep.

The three inclined holes were drilled in May, June and July, 1897, while the sump hole at Station 8 + 50, 10 L., the bottom of which was at Elevation —67, was in use. They were inclined in order to cross the vertical seams, and to better the chance of finding large erosions or caves. The results did not seem to indicate anything more extensive in the way of erosions than had already been found by the excavation for the first bottom formed between Stations 8 + 50 and 8 + 70, and it was some time later in the season that the excavation in its regular progress developed the location of the large cave to which attention has been called.

Grouting and General Treatment of the Rock Foundation.

Upon the completion of the rock excavation of any particular section of the bottom, which work included in many places a prolonged and tedious barring out and cleaning up of shaky or loose pieces of rock, the bottom was washed down and thoroughly cleaned by streams of water under a heavy head; and operations were then begun to clean out all erosions and open seams showing at the surface, and to trace them out as thoroughly and as far as possible by drilling numerous holes of varying depths in their vicinity.

All erosions and open seams were, as a rule, filled with Portland

FIG. 1.—NOVEMBER 20TH, 1896. MAIN DAM EXCAVATION. ROCK FACE, UP-STREAM SIDE.

FIG. 2.—FEBRUARY 26TH, 1897. MAIN DAM ROCK EXCAVATION AND MASONRY.

cement grout mixed with fine sharp sand of 1 to 1 or 2 to 1 mixture according as the grout was pumped or poured. In the case of the "cave," rubble masonry in mortar was used as filling in the large opening, while in some smaller erosions the spaces were thoroughly packed with small stones before the grout was poured in, and, in one or two exceptional cases, American cement was used for the grouting.

The drilling of holes in the rock bottom for grouting and searching purposes was begun as soon as the first section of bottom was excavated. Air drills were used, and were fed from the pipes used for drills at work on the general excavation. In all, about 1 700 lin. ft. of holes were drilled. They were about 2½ ins. in diameter at the rock surface, decreasing somewhat as the depth increased. These holes were of all depths up to about 16 ft., according to circumstances. Whenever it was found impracticable or inadvisable to pour or pump grout into holes or erosions before the adjacent masonry work was started, vertical pipes, generally 2 ins. in diameter, were placed in them and were then built around with the masonry up to such height as was necessary. In case holes or erosions showed a flow of water, such openings were also provided with pipes placed at proper inclinations to lead to some drain center or sump hole near by. In most cases the erosions, and in a majority of cases the drilled holes, were piped, as it was found more convenient and practicable to pump grout under heavy pressure into openings thus prepared and sealed and covered with masonry carried up to some convenient height. Old steam-piping was generally used for this purpose, and, while the diameter was generally 2 ins., other and larger sizes were sometimes provided, and, in the bottom, hard tile piping was at times used to carry water flows. In the case of the heavy spring at Station 5 + 95 large-sized, galvanized, riveted pipe was used to connect with the spring as the foundation masonry was built up, and at the cave at Station 7 + 70 it was found necessary to build into the filling masonry two 10-in. galvanized-iron suction pipes which were afterward filled by pumping grout.

Whenever grout was pumped a No. 2 Douglass deck pump was used. The grout was mixed by hand in boxes made for the purpose. The suction and delivery hose were each 3 ins. interior diameter, coupling to 2-in. hose at the pumps and also to a 2-in. nozzle, at the outer end of the discharge hose, made of a short piece of steam pipe. When

in use, this nozzle was either set well into the pipe leading to the channel to be grouted and carefully packed with waste and old bagging, or, in some cases, was coupled to the grout pipes by the use of screw threads and couplings, cut and furnished for that purpose. There were from four to six men at the pump handles, according to the resistance experienced in forcing the grout, and the pressure developed was sufficient in cases to burst the hose while still forcing appreciable quantities of grout to flow. As a preliminary to grouting any hole or section, care was always taken to flush the pipe and passage very thoroughly with water under a heavy head; a system of pipes leading from the Old Aqueduct, which was at an elevation of from 200 to 250 ft. above the main dam foundation, furnishing all the facilities for this purpose.

The contour plan, Plate XXXVIII, shows the position and extent of the various fissure erosions, cave and springs treated, together with the location and depths of all holes drilled in the process of tracing out; and also figures showing the number of bags of cement used in grouting at various places.

The first section grouted was between Stations 8 + 20 and 8 + 70. The search holes drilled had in many cases established a connection with the lines of erosions showing in the bottoms, and most of the holes were filled by the flow from holes adjoining. In this section 701 bags of Portland cement (175 bbls.) were used, mixed with sand (2 to 1). All erosions of any size were filled with small stones before being treated with grout.

The next section treated was along line 7 + 70, and included the cave which, after being cleared of gravel and having control of the water flow, gained by means of a 10-in. double Worthington pump (1 500 000 galls. per 24 hours), was filled with rubble masonry laid in Portland cement mortar. The sump-hole was established outside the line of the dam at about 27 R., and its location has been shown at the ends of the two suction pipes which, as stated, were left in and built around solid with masonry. The larger pipe was the one used in connection with the 10-in. pump. The second pipe was placed about 2 ft. higher than the other and was provided in order that drainage might be maintained in case the lower pipe should clog with sand or gravel during the filling up of the cave. A third pipe 3 ins. in diameter, reaching partly through the cave, was laid on the bottom below the

FIG. 1.—MARCH 2D, 1897. MAIN DAM ROCK EXCAVATION. UP-STREAM FACE.

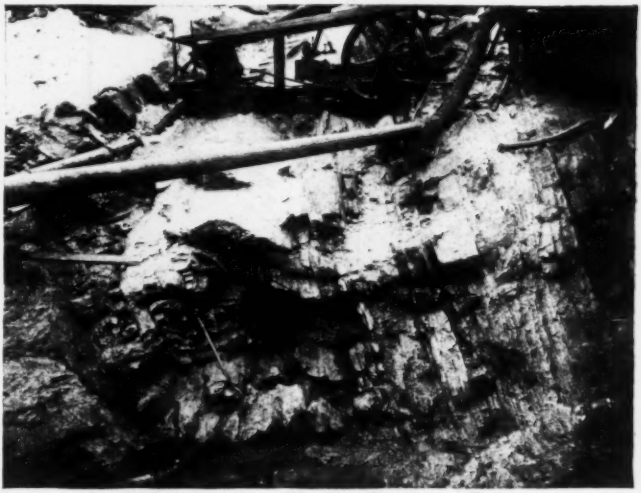


FIG. 2.—FEBRUARY 17TH, 1897. ROCK BOTTOM AND EROSIONS.

lower suction pipe and reached to the sump-hole. It was used to take slight flows from the walls and floor to the sump, and was also built in when the cave was filled. During this process the lower suction became clogged and the pump was connected temporarily with the upper pipe, while a stream of compressed air was blown through the 3-in. pipe. This freed the outer end of the lower suction which was again put in use, and no further trouble was had with it.

Fig. 7 shows cross-sections of the cave at various points. The section at 00 D. L. shows on the left the connection with the long oval-shaped pocket shown on the plan at Station 7 + 78. It also shows on the right a connection with a smaller pocket at Station 7 + 70. These pockets were comparatively deep, and plainly showed erosion between the solid stratum forming the cave roof and the somewhat softer stone on each side. The section at 125 R. shows the average cave section inside the masonry lines. At 25 R., just outside the up-stream line, the cave abruptly enlarges, reaching nearly to the rock surface, while at 37½ R., the section is somewhat smaller apparently, although it was not free enough of gravel and sand to show that clearly.

As the cave was cleared out it was heavily timbered in the roof for the protection of the workmen from the possible fall of detached pieces of rock, and, when 27 R. was reached, a timber bulkhead about 4 ft. high was built to retain the gravel slope lying in the fissure beyond. It was also of use in forming the outer wall of the sump-hole which was located on the extreme right of the cave where the roof was low. A shaft 4 to 6 ft. square and about 6 ft. deep, between Stations 7 + 69 and 7 + 75, was sunk to reach the roof of the enlarged cave section which, at this point, ran back about 10 ft. from the outer neat line of the dam, re-entering for that distance over the roof of the cave proper and near the rock surface. The masonry filling began close to the bulkhead line, 27 R., and was carried up on that line vertically to the surface through the 4 x 6-ft. shaft which was excavated for that purpose. As this filling progressed in the cave, working toward the center of the dam, the timber was gradually removed with the exception of two 8 x 12-in. range timbers which extended throughout its whole length and which were built in. A small shaft was sunk into the roof of the oval-shaped pocket on the left, at Station 7 + 78, and when the cave masonry below had been built to the general roof level this pocket was filled with small stones and then grouted, taking

forty-eight bags of cement (2 to 1 mixture). The smaller pocket on the right was packed with stones from below and grouted through an inclined drilled hole 12 ft. deep, taking eight bags of cement (2 to 1 mixture). Other inclined holes were drilled in this vicinity—12 to 18 ft. in depth—in a search for further cavities.

To the pump ends of the suction pipes as they were built in, reducers and 2-in. iron pipes were finally attached, and the water from the sump-hole outside was allowed for a long time to flow through and was conducted to a temporary sump-hole near the center of the work, while the masonry was gradually built up. As these pipes were raised higher, this flow finally stopped, as the back-filling on the up-stream side then in place was not sufficient to prevent the flow from finding an outlet in the up-stream sump to the south. These suction pipes were grouted when the masonry and connecting pipes had been raised about 40 ft., 72 bags of Rosendale cement (1 to 1 mixture) were poured into the lower and longer pipe, filling it, then 42 bags of Rosendale cement (1 to 1) were partly poured and partly pumped into the upper and shorter pipe. The pumped material was forced up through the back-filling on the up-stream side and this caused a temporary stopping of the experiment. Some time later (about a year), 11 bags more were pumped in, and the hole was blocked, no sign of this grout showing this time in the up-stream back filling, which, in the meantime, had been carried up much higher. In the narrow, eroded seam lying along the line from Station 7 + 32, up stream, to Station 7 + 58 down stream, 300 bags (75 bbls.) of cement were used, the grouting showing at least connections between adjacent erosions and search holes as the seam was pumped full.

Beyond, between Stations 6 + 80 and 7 + 10, and partly including a bottom which was hard and solid, but full of open seams and erosions, and distinguished by some solid masses which rose above the general surface, a great many pipes were used, and a large quantity of grout was pumped in.

The next bottom section, covering the lowest point reached for a foundation, was drilled and treated as usual, but the extreme low bottom on the up-stream side took but little grout except along the line of seams from 20 R. to 30 L. near Station 6 + 50.

On the down-stream side will be noted some lines of erosions into which considerable grout was pumped.

FIG. 1.—JUNE 25TH, 1897. ROCK BOTTOM, SHOWING ERODED SEAM.

FIG. 2.—AUGUST 12TH, 1897. ROCK EXCAVATION, LOOKING SOUTH FROM STATION 6 + 00, 60 L.

But little grout was used beyond this bottom until the eroded seam between Stations 5 + 30 and 5 + 40 was reached, as the spring hole at Station 5 + 93 was not treated by grouting.

The erosions along the 5 + 40 line were drained at a sump-hole at 39 L. during the excavation, while the bottom masonry was being laid and the drilled holes and erosions were being piped. A well was therefore gradually built up at this point, reaching a depth or height of about 20 ft. before the grouting work was started. The holes in the seam between the well and the up-stream side were grouted by pumping before the well had reached this height, as there was no connection between them, the water in the well coming wholly from the other direction. When the well was ready the drainage pump was taken out and the drainage was maintained by a pump attached to the 5 in. pipe shown at 165 L. This pipe was just outside the down-stream toe line of the dam, and had been placed and used for a drainage well while the rock excavation in its vicinity was being made.

The main well hole, as it was built up, in places had its down-stream face built of stones laid dry, in order that seams in the adjoining rock might not be shut off from the grout later, as well as to allow free passage of the water to the suction pipes. A 2-in. pipe was also built into this well, reaching to its lowest point and connecting there with seams in the rock.

The well was filled with 80 bags of Portland cement (1 to 1 mixture) poured in, and it was evident from the water which was forced from pipes nearby, notably at 54 L., that the grout was reaching the seams and passages in that direction. As the grout was poured the well was gradually filled with small stones collected for that purpose. After no more grout could be poured 4 bags of cement (1 to 1 mixture) were pumped into the pipe placed in its corner. The grout pump was then tried in each pipe in turn working toward the down-stream side. The grout was forced gradually into the 5-in. pipe, the pump at which was stopped when the grout trace became marked. This pipe was filled, so that the water flow ceased through it, by pumping at the spring holes near by at 150 and 158 L. Later, after the pump was disconnected, it was completely filled by pouring 5 bags of cement (1 to 1 mixture) into it at the top. By this time, the water which had flowed along this seam was blocked off entirely and had forced its way up to

the surface of the down-stream gravel slope at an elevation considerably above the top of the 5-in. pipe.

Beyond this seam, little grouting was found necessary until Station 4 + 60 was reached, where the seam developed in the course of the rock excavation showed some traces of erosion on the solid vertical face left in on the north side. The nearly horizontal open seam reaching under this vertical face showed a few springs, and at two places, 43 and 126 L., 18 and 30 bags of cement (1 to 1 mixture), respectively, were pumped in, and smaller quantities at other places. The spring at 43 L. was filled and "X"d at three points between 25 and 31 L. The grouting nearer the down-stream side gradually drove the water outside the masonry limits. Six holes, 12 to 14 ft. in depth, were drilled on the line of this seam near the up-stream side, but further traces of it were not found. A number of pipes were placed between 18 and 45 L., Station 4 + 10 to Station 4 + 40, above and along very narrow but somewhat open seams in masses of solid white rock, and, as the bottom masonry was laid, these pipes were connected by covering the seams with small spawls laid dry. The pumping afterward done indicated free flowing between the pipes, and a considerable portion of the grout must have been used to fill channels thus provided.

The same remarks apply to the piped seams from 40 to 60 L., Station 3 + 50 to Station 4 + 10, where the open seams were in most cases so fine that they could have taken but little of the grout pumped. A wider seam at Station 3 + 75, 0 to 30 L., was piped and took an appreciable quantity, as is shown on the plan. There were no signs of erosion there.

The drilled holes, 16 to 18 ft. deep, between Stations 3 + 30 and 3 + 60, practically took no grout. Neat cement was pumped into them, and the amount taken was only about enough to fill the holes.

They were drilled, as the rock, though compact and fairly solid, was full of short heads and tight seams, and it was thought best to make sure that the seams were no looser below.

The Spring at Station 5 + 93.

It was at first proposed to grout this spring, as well as the others, but circumstances led finally to a different course of treatment in this case.

The flow from this spring was heavy. When first uncovered it was curbed with a bag dam and piped as shown in Fig. 2, Plate XLI. The

pipe was 8 ins. in diameter, and the flow was sufficient to back up against the pipe at the entrance. The inclination of the pipe, however, helped the flow, and at its lower end the pipe was about half full. It was unfortunate that no gauging of this flow was ever made, but, with many other springs and flows about, it was overlooked.

The flow was carried through this pipe for some months until the masonry work had reached the spring level, when it was taken by a 12-in. iron pipe laid in the masonry to the sump-hole by this time established near by, outside the up-stream face which had been carried up to about 30 ft. above the lowest point of the rock bottom. Later, another 12-in. pipe was laid from a slightly higher elevation through the masonry outside the face of the dam. This pipe was continued with a 90° elbow, and, after plugging the lower pipe, short vertical lengths of pipe were added to it by which the point of discharge was gradually raised as the masonry forming the well above the spring was carried up ahead of the discharge pipe. This arrangement also allowed the back-filling against the up-stream face to be carried on conveniently without impeding the flow of the spring or filling it with earthy material. The head of the spring was reached at about Elevation + 20, when the flow, which had been gradually diminishing, ceased. This was about 83 ft. above the rock bottom of the spring hole and 74 ft. above the outlet pipe which by this time had served its purpose for 15 months while the masonry was building. The section in Fig. 9 shows the above-described features, as well as the partial location and trend of the open fissure leading from the spring hole. This fissure showed at the well hole a considerable section; its direction was down stream with a downward horizontal dip. Viewed from above the well hole, the bottom was full of well-washed gravel stones of comparatively small size. As the passage was always full of water flowing swiftly, it could not well be explored, but the line of drilled holes, shown on the contour plan, from 7 to 17 ft. in depth, was used to locate its position and direction as far as practicable. Five of these holes reached the fissure or connecting seams and were piped with 2-in. pipes until the head of the spring was reached. The additional sections shown in Fig. 9 are from the results of these borings. When the well had been built to Elevation — 2, it was decided to seal it, as to carry it higher involved unnecessary complications with the masonry work. It was 4 ft. square. A 3-in. iron pipe

was placed in one corner and reached nearly to the bottom. A large flat stone was lowered, and by tag and guide ropes so placed as to partly cover and shield the 12-in. pipe opening. Its position was directed by sounding with plummet and wire line, and two trials only were necessary to place the stone properly. The well was then filled up to Elevation — 47, about 8 ft. above the outlet pipe, with clean spawls, the larger sizes being placed on the bottom and gradually diminishing to the size of concrete material at the top. These stones were lowered to place in a box built to fit the well, and with a bottom which could be tripped open. The same box was used to continue the filling with concrete, Portland cement in the proportion of 4 gravel, 2 sand and 1 cement being used. The dump box had a capacity of 18 cu. ft. The first batch of concrete was mixed dry and was placed in the box on a sheet of canvas which covered the bottom and sides of the interior, and formed a tight bottom for the concrete mixture when it was dumped. The dumping of the concrete was continued diligently until it had risen about 13 ft. This work had not in any way disturbed the flow through the outlet pipe, which showed no discoloration due to cement or gravel, and examination of the 3-in. pipe in the well corner indicated that the sealing was complete, as the displaced water in the well was overflowing at the top, while the water in the 3-in. pipe remained stationary. This was further shown the next day when the well was bailed out nearly dry to facilitate its further filling with concrete. When filled the masonry work above and around the well was resumed, but the 3-in. pipe was carried up with the others until the head of the spring was reached.

One of the four 2-in. pipes furthest away from the well hole took water freely. They were about 90 ft. long. The water from the spring rose in the line of pipes to Elevation 20 ±, about 10 ft. below their tops. It was a question whether grout could be poured successfully through so much water, and in the first pipe tried, 17 L., the grout clogged half-way down the pipe, owing, apparently, to some roughness due to carelessness in joining the sections of the pipe. This was washed out by a flow of water pumped through a $\frac{1}{2}$-in. pipe, and the pipe was cleared.

It was then suggested that an effort be made to fill this erosion with plastic clay by driving through the pipes. This suggestion came from the contractors, who had used clay to fill cavities under some-

what different conditions. Arrangements were made to try this method, and a small pile-driver with a 2 000 lb. hammer was set up over the 2-in. hole at 32 L.

A piece of 3-in. steam-pipe, about 6 ft. long, was used at first as a receiving cylinder, and it was connected to the 2-in. pipe which projected above the masonry about 1 ft., with a strong reducing coupling. The follower used was a $2\frac{7}{8}$-in. steel rod, of the length of the cylinder, turned to fit closely its full length. It was welded at its upper end to a lengthening rod, slightly smaller in diameter, which was fastened at its upper end to a wooden cross-head, designed to work in the guides of the pile-driver and to take the blow of the hammer.

Blue clay of good quality was used, and was made plastic with water and a thorough working and pounding into boxes 10 ins. deep and of a size to hold 10 cu. ft. These boxes were limited in dimensions simply for convenience in keeping record of the clay used. The clay was then cut into "sausages" 10 ins. long and $2\frac{3}{4}$ ins. in diameter by shovels which had been bent and sharpened for the purpose. As often as the shovel was used to make a cut it was dipped in a pail of water in order to lubricate its surface and free itself for the next cut. The sausages were passed to the pile-driver in boxes holding 50 lbs., and the amounts of clay thus determined were used for the purposes of a record, and later, were reduced to cubic yards.

The first hole to be tried was one which took water very slowly. On starting the clay driving, the machine worked very satisfactorily, but the clay drove hard and only $3\frac{1}{8}$ cu. ft. were driven in all; probably not much more than enough to fill the pipe, which was about 90 ft. long, and the fissure at its bottom. The next hole tried was at 17 L. It was also a slow water hole. Into this 332 lbs. of clay were driven, with the 3-in. cylinder, everything working hard, and an 18-in. drop of the hammer being necessary. It was then decided to change the 3-in. cylinder for a 2-in., with, of course, a correspondingly smaller piston. This piston proved to be somewhat loose in its fit, but the driving continued to be very hard and only 37 lbs. of clay in addition, were put into this hole.

The driving was then resumed at the hole at 10 L., the 3-in. cylinder being again used while a new piston was being fitted for the 2-in. cylinder. The hole took water very freely, and 200 lbs. of clay were driven easily, using a 1 ft. drop. The next 200 lbs. went harder and

required a 3-ft. drop before it was all in. A change was then made to the 2-in. cylinder and piston, but the difficulty of driving seemed to increase, although a drop of 4 ft. was given the hammer. With this, only 58 lbs. more were driven in, and then the pipe split between the coupling at the foot of the cylinder and the masonry. After repairing this break driving was resumed, with a 4-ft. drop of the hammer, for 2 hours, and very little clay was gotten in, when it was noticed that everything worked more easily and a drop of only 2 ft. was necessary. In the course of another 2 hours less drop was used and cracks began to be noticed in the surface of the masonry radiating from the pipe in use. Further effort confirmed this, and, on the next day, observations with level and transit, during a short period of driving, in which 50 lbs. were easily put into the hole, showed a distinct and appreciable rise in the masonry surface. In all, about 150 lbs. were driven in this pipe after the split had been repaired.

Leaving the upheaved masonry to be investigated later, the driving was transferred to the 3-in. pipe which had been built into the well at Station 5 + 93 as it was filled up. There was no question about a free flow through this pipe and a plumb-bob dropped in readily found bottom in the well below the known elevation of the bottom of the pipe. However, in view of the difficulty experienced in driving clay through the 2-in. pipes already tried, it was thought advisable to make a test to determine to what extent, if at all, skin friction interfered with the passage of the clay.

The 3-in. cylinder was therefore connected by a quarter turn with 97 ft. of 3-in. pipe resting on the masonry surface. At the further end another quarter turn and a 2-ft. length of pipe gave opportunity to fit a poppet valve, which was set at 34 lbs. per square inch. The pipe was then filled with water, and clay was gradually forced in from the cylinder end. It was found to require no force beyond the weight of the hammer without impact and the water was forced through the poppet valve as fast as the clay was pushed in at the outer end. Later, 28½ ft. of 2-in. pipe and 20 ft. of 1½-in. pipe were joined to the 3-in. pipe and the valve fixed at the outer end. Under these circumstances it took a drop of about 14 ins. to force clay to the extreme end and through the valve. An examination of the clay as it was forced from the ends of the various sizes of pipes showed clearly that, under even a very slight compression, the water is driven to the clay surface next

the interior surface of the pipes and acts as an efficient lubricator, the skin friction amounting to practically nothing.

Driving was then resumed at the 3-in. pipe; 3 250 lbs. of clay were forced in, only the weight of the hammer being needed on the first day.

Appended is an abstract from the log of the clay driving, following the work above noted:

"Tuesday, December 27th, 1898.—4 500 lbs. driven in 3-in. pipe; hammer only used; no impact to force clay. No evidence of clay in 8-in. pipe outside of masonry, though distinct tremor in water was noticed during driving. Water in 8-in. pipe remained at constant height, about 6 ft. from top of pipe. Elevation, 18.4.

"Wednesday, December 28th, 1898.—4 400 lbs. put in. About 8.45 A. M. water in 8-in. pipe had risen to within 20 ins. of top, and rose 5 ins. at each stroke of piston, falling back to old level after stroke. About 10 A. M. water began to flow over edge of pipe at each charge, but settled back below edge after stroke. About 12.00 M. water ceased to settle back. About 3.00 P. M. began to run in a small stream after stroke had been made. Plumbed pipe, but found no clay in it. Distance, measured from top down, 81.04 ft.

"Thursday, December 29th, 1898.—A small stream of water was flowing from 8-in. pipe when work started. 5 000 lbs. driven to-day. Plumbed 8-in. pipe as follows:

9.00 A. M.	Distance 81.04 ft.	—No clay in pipe.
2.50 P. M.	" 78.62 "	—2.42 ft. of clay in pipe.
3.50 "	" 78.08 "	—Rise of 0.54 ft. of clay with 750 lbs. put in between 2.50 and 3.50 P. M.

"At 2.50 P. M. water flowing from pipe was strongly colored with clay, which gradually cleared, and about 4.00 P. M. no trace of color could be detected in water flow.

"Friday, December 30th, 1898.—Clay driven 5 200 lbs. Plumbed 8-in. pipe as follows:

9.20 A. M.	Distance 77.28 ft.	—Rise 0.80 ft.	1 300 lbs.	clay.
11.20 "	" 76.28 "	— " 1.00 "	1 250	"
2.20 P. M.	" 71.94 "	— " 4.32 "	1 850	"
4.00 "	" 70.88 "	— " 1.06 "	800	"

"At 11.20 A. M. water was running clear from 8-in. pipe and reduced in amount over yesterday, with very slight acceleration in flow, when charge was driven.

"Saturday, December 31st, 1898.—3 750 lbs. driven. Plumbed 8-in. pipe as follows:

9.20 A. M.	Distance 68.85 ft.	—Rise 2.03 ft.	2 150 lbs.	clay.
10.20 "	" 68.15 "	— " 0.70 "	1 000	"
11.20 "	" 67.80 "	— " 0.35 "	1 000	"

"Water flowing from 8-in. pipe is clear and reduced in amount over yesterday, shows very slightly the effect of each charge. Total clay driven through 3-in. pipe to date, 26 100 lbs.

"Wednesday, January 4th, 1899.—4 000 lbs. clay driven. Plumbed 8-in. pipe as follows:

12.50 P. M.	Distance 67.70 ft	1 000	lbs. clay.
2.00 "	" 67.60 "	1 000	"
3.10 "	" 67.55 "	1 000	"
4.45 "	" 67.50 "	1 000	"

"Water flowing from 8-in. pipe has decreased slightly since December 31st; runs clear and shows, very slightly, effect of driving charges. The weight of hammer only, continues to be required to drive clay down.

"Thursday, January 5th, 1899.—5 300 lbs. of clay driven. Weight of hammer only, required; measurements taken on 8-in. pipe as follows:

9.55 A. M.	Distance 67.30 ft	1 000	lbs. clay.
11.25 "	" 67.22 "	1 000	"
1.05 P. M.	" 67.15 "	1 000	"
2.15 "	" 67.07 "	1 000	"
4.20 "	" 67.00 "	1 000	"

"Saturday, January 7th, 1899.—5 100 lbs. clay driven; no change in measurements taken in 8-in. pipe:

9.10 A. M.	Distance 67.00 ft	1 000	lbs. clay.
11.25 "	" 67.00 "	1 000	"
1.25 P. M.	" 67.00 "	1 000	"
2.40 "	" 67.00 "	1 000	"
4.10 "	" 67.00 "	1 000	"

"Flow of water from 8-in. pipe clear and constant; shows no effect of charge; driving with weight of hammer only.

"Monday, January 9th, 1899.—6 200 lbs. put in; no change in measurements in 8-in. pipe taken every 1 000 lbs. All show clay at distance from top of pipe of 67 ft., or at elevation—32.44. Flow of water shows decided increase over January 7th, and runs steadily and clear, showing no effect of ramming. Weight of hammer only, required.

"Tuesday, January 10th, 1899.—4150 lbs. driven. No change in measurement in 8-in. pipe. All show clay at 67 ft. down from top of pipe. Weight of hammer only used, no impact. Water flowing from 8-in. pipe shows slight increase over yesterday; runs clear.

"Friday, January 13th, 1899.—Clay ramming resumed to-day. Total driven, 3 750 lbs. Measurements taken in 8-in. pipe as follows:

10.20 A. M.	Distance, 67.00 ft	1 000	lbs. clay.
12.00 M.	" 66.90 "	800	" "

"The first 1 000 lbs. drove easily; requiring weight of hammer

only, but driving seemed to stiffen until, at end of next 800 lbs., a slight drop of hammer, about 5 ins., was required.

12.40 P. M. Distance, 66.85 ft.—200 lbs. clay; still slight drop of hammer.

2.30 " " .... —250 lbs. clay. Water in pipe rose suddenly, discharging over edge in quite large volume, indicating clay has been forced upward suddenly; and after this, driving becomes easier, weight of hammer only required at:

2.45 " " 60.90 ft.—Rise 5.95 ft., 250 lbs. clay. Water was discharged as each cylinder full of clay was forced in, and continued to flow between strokes also, at the rate of about 13½ galls. per minute until 3.15 P. M., when flow stopped, except as charge was driven.

3.50 " " 46.95 ft.—Rise, 13.95 ft. 500 lbs. clay.

4.40 " " 37.90 ft.— " 9.05 " 500 " "

"Saturday, January 14th, 1899.—4 250 lbs. clay driven. Measurements taken as follows:

9.50 A. M. Distance, 35.01 ft.—Rise, 2.80 ft. 500 lbs. clay. Water in pipe rises about 2 ins., when charge is driven, dropping back again to old level, but does not flow out of the pipe. Some water noticed coming up through back-filling around pipe, evidently from leaky joint.

10.45 " " 33.70 ft.—Rise, 1.40 ft. 500 lbs. clay.

11.20 " " 32.90 " — " 0.80 " 500 " "

11.55 " " 32.50 " — " 0.40 " 500 " "

1.10 P. M. " 32.20 " — " 0.30 " 500 " "

2.15 " " 31.75 " — " 0.25 " 500 " "

Driving now began to stiffen up and at 2.45 a slight drop of hammer, 6 ins., was required, continuing for balance of day.

3.10 " " 30.50 ft.—Rise, 1.25 ft. 500 lbs. clay.

3.55 " " 28.95 " — " 1.55 " 500 " "

4.35 " " 28.65 " — " 0.30 " 250 " "

"At this time water in pipe ceased to show any effect of driving charge. This P. M. the clay exhibited remarkable elasticity, sometimes forcing the piston back 3 ft. after driving charge.

"Monday, January 16th, 1899.—Total clay driven, 3 400 lbs. Required impact of hammer to force clay, limiting the height of stroke to 6 ins. About 45 strokes required for the cylinder, which is 6 ft. long. The clay is stiffening gradually and losing some of its elasticity, not springing back as much after each stroke. Measurements in 8-in. pipe as follows:

7.50 A. M.	Distance,	28.55 ft.	250 lbs. clay
8.50 "	"	28.55 "	500 " "
10.00 "	"	28.55 "	500 " "
12.30 P. M.	"	28.55 "	500 " "
1.35 "	"	28.55 "	500 " "
3.50 "	"	28.52 "	500 " "
4.15 "	"	28.65 "	500 " "

"Tuesday, January 17th, 1899.—Continued driving in 3-in. pipe until noon; 1 600 lbs. driven. No rise in 8-in. pipe to-day. Clay gradually stiffening in 3-in. pipe until it requires about 90 strokes, none over 6-in. drop, to force down a cylinder full. Shifted over and started driving in 2-in. pipe at Station 5 + 97.5, 3 R., at 3.35 P. M. Weight of hammer only used, carrying clay down very slowly; 200 lbs. of clay put in.

"Wednesday, January 18th, 1899.—Continued at 2-in. pipe, Station 5 + 97.5, 3 R. 200 lbs. put in, with weight of hammer only. At noon this ceased to have effect, and a few light blows, none greater than 6 ins., were tried. Shifted pile-driver back again to 3-in. pipe in P. M. By means of water-jet, clay was removed from 8-in. pipe to a depth of about 40 ft. in order, if possible, to start clay rising in pipe again when ramming should be resumed.

"Thursday, January 19th, 1899.—Jetted out 8-in. pipe to a depth of 40.8 ft. from top when jet stopped short and seemed to bring up on small spawls or gravel. Resumed ramming at 11.00 A. M. in 3-in. pipe, using about 5-in. drop of hammer. Clay working very stiff, requiring about 240 strokes to drive charge (6-ft. cylinder); 500 lbs. put in. Total rise in pipe, 0.5 ft., from 40.8 to 40.3. Ordered clay driving stopped.

"Friday, January 20th, 1899.—Drove 100 lbs. of clay in 3-in. pipe to-day, somewhat easier than yesterday. This was done for the information of members of the American Society of Civil Engineers who visited the work to-day.

"Saturday, January 21st, 1899.—Resumed jetting in 8-in. pipe in endeavor to increase depth of jetting to at least 20 ft.

"Monday, January 23d, 1899.—Succeeded in jetting out 8-in. pipe to a depth of 60 ft. below top. At this point jet pipe brought up on

FIG. 1.—AUGUST 10TH, 1897. SEAM AT STATION 4 + 55, LOOKING EAST.

FIG. 2.—OCTOBER 22D, 1897. GENERAL VIEW FROM BERM, AT ELEVATION 115.

what seemed to be a bed of gravel. The flow from the 8-in. pipe increased when this point was reached.

"Tuesday, January 24th, 1899.—Resumed driving in 3-in. pipe; 350 lbs. put in by means of short drops of hammer, none over 6 ins.; 320 short drops required to force down piston (6-ft. cylinder). No rise of clay in 8-in. pipe or any indication of so doing. Ordered clay driveing stopped."

It seemed to be apparent that the cavity at the foot of the 3-in. pipe was well filled with clay, and it was probably due to some obstruction in the 8-in. pipe, such as spawls or gravel which had been forced in with the clay, that there was no longer any rise in the vertical section. The total amount of clay used in this driving is as follows:

2-in.	pipe,	Station 6 + 02,	32.4 L.	.....	395 lbs.
2-in.	"	" 6 + 03,	17.7 L.	.....	369 "
2-in.	"	" 6 + 02,	9.7 L.	.....	1 012 "
3-in.	"	" 5 + 95,	12.5 R.	.....	64 775 "
2-in.	"	" 5 + 97.5,	3 R.	.....	400 "
		Total.		.....	66 951 lbs.

66 951 ÷ 113 = 592.5 cu. ft. or 21.9 cu. yds.

The total amount of clay thus driven into the pipes and cavity was nearly 22 cu. yds. The 60 ft. in depth of the outer 8-in. pipe which had been jetted out, was afterward filled with clay and gravel rammed in by hand, and the flow through it was stopped.

The work of tearing out the ruptured masonry due to driving clay through one of the 2-in. pipes was immediately started, and in all about 130 cu. yds. were taken out in following the fissures and cracks until they wholly pinched out. It was assumed in the beginning that the trouble lay in the joint between the upper or surface course of masonry at this point, which had been laid in Portland cement, and the masonry below, which had been laid in slower setting natural cement, and this proved to be the case; as it was found on investigating around the pipe from which the cracks radiated that its upper section and length, of something more than 5 ft. in all, had not been joined or coupled with the section below when placed, and that there was a space of 1 in. or more between the ends of the two pipes, which had allowed the clay, under the influence of the very hard driving, to force its way into the partly set mortar which surrounded the joint.

The course of Portland cement mortar was about 3 ft. thick. It was found that this upper joint of pipe had lifted probably 1¾ ins. from the joint below at the point of coupling. This must have taken place at first when the heavy ramming caused the rupture in that part of the pipe to which the clay cylinder was coupled, and which projected above the top of the masonry. The length of this projecting part was about 1 ft. and the parting of the joints was, of course, at the lower end and about 4 ft. 4 ins. below the masonry surface.

Adjacent to and on the level of this open joint in the pipe was the bed joint of a stone laid in Rosendale cement mortar. The stone was about 5 ft. long, 20 ins. wide and 15 ins. in thickness. The clay was found to have been forced between the under surface of the stone and its mortar bed. The clay bed was about 1½ ins. thick, and, continuing beyond the base of this stone, it rose through the joints along the sides, finding its way then along the top of the course of which the stone just mentioned formed a part, and lifting the course above, which had been laid in Portland cement mortar, and was fairly well set. The first stone mentioned was the only one laid in Rosendale cement that was found to have been disturbed in its bed, and the main crack was everywhere along the junction of the two cement mortars.

The longest horizontal radius through which the clay was found to have worked was about 5½ ft. around the pipe, and it worked up vertically through the mortar joints, and especially along the pipe, about 3 ft. The vertical cracks showing in the masonry surface were traced in some directions for 12 ft., where the width showed about 0.005 ft.; but the upper masonry course was taken out to a considerably greater distance toward and to the up-stream face of the dam, where the seepage of water through the exposed bed joints of one or two stones in this upper course indicated that the horizontal crack had extended with no vertical surface crack above to call attention to it. The clay bed, thus forced under the masonry, was found to be fan-shaped, extending 5½ ft. from the pipe in one direction and about 4 ft. sideways on each side. In the other direction its course was arrested by a large stone which offered no mortar joint that could be followed. The bed varied from 1½ ins. to ½ in. in thickness. It was found to consist of an aggregate of very thin laminations which showed clearly throughout the extent of the bed and indicated the extremely gradual way in which the rupture was produced.

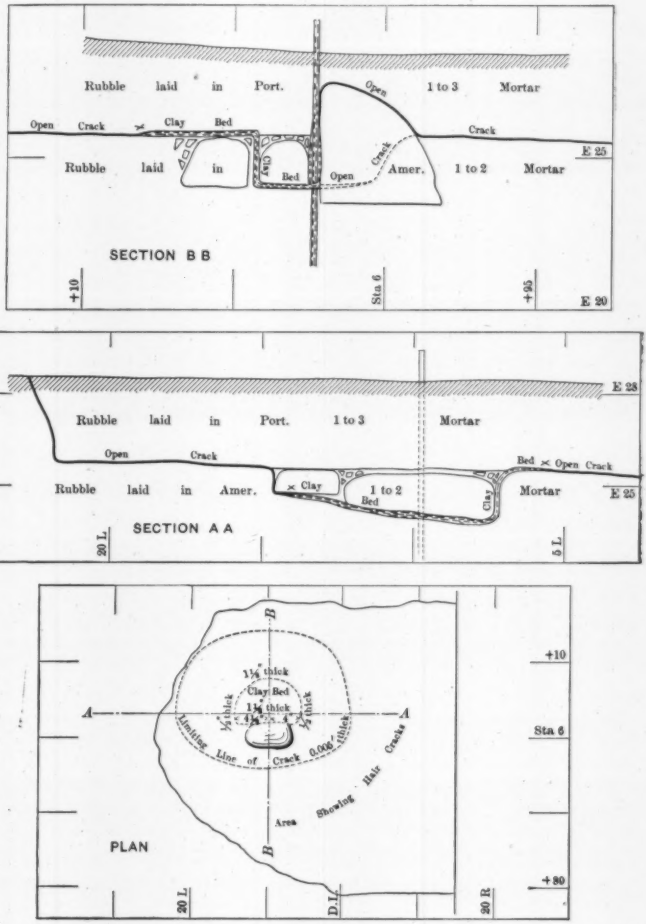

Rubble laid in Port. 1 to 3 Mortar
Open
Open Crack
Clay Bed
Crack
E 25
Rubble laid in Clay Bed Open Crack Amer. 1 to 2 Mortar
SECTION B B
+10
Sta 6
+95
E 20
E 28
Rubble laid in Port. 1 to 3 Mortar
Open Crack
Bed Open Crack
Rubble laid in Amer. Clay 1 to 2 Bed Clay Mortar
E 25
SECTION A A
20 L
5 L
B
1¼" thick
Clay Bed
1¼" thick
½" thick
½" thick
A
A
Limiting Line of Crack 0.005" thick
Area Showing Hair Cracks
B
+10
Sta 6
PLAN
+80
20 L
D.L.
20 R

FIG. 11.

Fig. 11 shows in plan and sections the area of the masonry which had to be taken up, as well as the location and extent of the clay bed, the cracks and the particular joints and stones, sketched during the work of rectifying the damage done.

The Main Dam Foundation Masonry.

The laying of the foundation masonry began on May 28th, 1896, in the bottom at Station 8 + 50, as soon as a sufficient area of bottom was ready to warrant it, and by the end of that season nine gangs of masons were at work. This involved the use of eleven or twelve derricks to allow for time lost in shifting derricks as well as for changing gangs from one point to another, owing to frequent changes in the location of sumps and subsidiary pumps which the maintenance of drainage made necessary. In the following season (1897), the number of gangs was increased to 17 on the foundation, as the season progressed, and the total number of derricks in use, including those on the side slopes for passing material from the tracks, was about thirty.

The type of derrick in general use is the "stiff leg" derrick. Having no guys, these derricks did not interfere with the cable service and they were easily moved by means of the cable, without being separated from stiff legs and platforms.

The setting of the first stone in the main dam foundation is shown in Fig. 2, Plate XXXVII. The bottom courses were laid in Portland cement (2 to 1), the vertical thickness of this work, varying from 4 ft. up, depending upon circumstances and particularly upon the amount of seepage through the fissures in the rock and the work necessary to temporarily dam up and divert such flows until the masonry was old enough and high enough to enable them to be blocked off permanently.

The rock bottom was, in all cases, very thoroughly washed and cleaned with brushes and brooms, and was then "painted" with a grout of neat Portland cement, applied with brushes, and which was allowed to set before work was done upon it. This grout was for the purpose of filling all small, fine, open cracks, seams and erosions, which were not of sufficient size or importance to warrant special treatment with the grout pump or box, and about 356 bbls. of Portland cement were used in this way on the main dam foundation and 14 bbls., up to date, on the core-wall and overflow foundations.

Owing to the extreme unevenness of the rock bottom as prepared,

FIG. 1.—MARCH 3D, 1899. DAM, FROM HILLSIDE NEAR SPUR ROAD, LOOKING SOUTH.

FIG. 2.—JUNE 22D, 1898. UP-STREAM SIDE, STATION 8 + 70 ±, SHOWING JOINTS RAKED OUT FOR POINTING.

the numerous spring holes which had to be temporarily dammed in order to divert their flows until such time as they could be choked off, and the number of grout pipes to be placed, and in some cases kept in place for months before the grouting could be done, the first season's work on the bottom masonry was done under difficulties, and the general progress up to January 1st, 1897, was between Station 7 + 12 and Station 9 + 62, covering the full width of the dam and varying in depth or height above the bottom from 10 to 40 ft. In all, about 37 000 cu. yds. were laid.

In the following year, the masonry was extended over the whole bottom, with the exception of a narrow strip near the river wall at Station 10 + 00, used as the foundation of a trestle work in connection with the supply tracks, and a comparatively small area in the center of the wall, at about Station 7 + 50, which, during the latter part of the season, was used as a sump-hole for the main pumps, the surrounding masonry forming the sides. During that year the amount of masonry laid in the foundation was about 115 000 cu. yds., and at certain points it had risen to a considerable height, particularly over the points at which the work was started during the preceding season. The width of the foundation to be laid was about 200 ft., and the derricks were arranged in batteries of four abreast across the line of the dam, the plan being to build in racks to a convenient height and then to move the derricks forward in batteries. In this way successive racks or steps were gradually formed for the full width of the work, varying from 17 to 15 ft. in height and from 35 to 40 ft. in width or depth, horizontally, with the derricks moving from the ends toward the middle of the foundation. This arrangement is shown in Fig. 2, Plate XLIV, but, owing to circumstances, it was not until the end of the second season's work that it could be said that the plan had been fully developed and put in complete working order.

The faces or step courses of these racks were limited to rises of 3 ft., with about the same treads, making the slope of the rock nearly 1 : 1. Care was taken to avoid long straight joints across the dam, between successive racks, by varying the lines of their faces at intervals with "scallops" or heavy "returns."

By the end of the second season, the main foundation had risen high enough above the bottom to be drawn in to its neat lines at all points and to afford a parapet which retained the wash from

the earth slopes and enabled the refilling work to be started. On the up-stream side, the masonry face was drawn to its neat lines as soon as practicable upon leaving the rock bottom, though this involved a considerable depth of re-fill between the masonry and rock face, particularly at the deeper points where the rock was disintegrated and had broken back of the excavation lines.

On the down-stream side the masonry toe was built solid to the rock face up to the surface, when it was drawn in to the neat lines planned. This junction with the rock face was made still more compact, as noted in the description of the "grouting," by filling with grout the eroded seams showing in the rock face as the toe masonry was built up.

As above stated, the courses adjacent to the rock bottom were laid in Portland cement, a 2 to 1 mortar mixture being used. The special purpose was to obtain a quick-setting mortar and thus avoid, to as great an extent as possible, any wash or trouble from seepage and flows through the bottom which had been choked off. Above these courses, and for the great bulk of the warm season's work, American cement, mixed 2 to 1, was used. During the winter months, Portland cement, 3 to 1 mixture, was substituted for the American cement, and work was carried on steadily on pleasant days when it was not too cold.

Care was taken to lay no masonry on days when the temperature was steadily below the freezing point, and on cold nights and mornings brine and warm water were used in mixing mortar, and the sand during the whole season was heated and dried in large boxes furnished with steam coils arranged for that purpose. Care was also taken to cover fresh work at night with brine, salt and canvass, and to thoroughly clean its surface and joints in the morning with steam and hot water in order that all frozen dirt and mortar scale might be removed. All stones and spawls used in cold weather were also thoroughly cleaned and washed, and thawed out with hot water and steam; pipes for both being provided for each gang of masons employed.

The stone used for the rubble masonry is quarried from a rocky hillside in the Valley of Hunter's Brook, a tributary of the Croton River, at a point about 2 miles above the dam. This stone is classed by geologists as "gabbro" rock, or, commercially, as a dark colored granite, although it is without quartz and has a large amount of hornblende in its composition. It is very hard and tough, as well as heavy, and

FIG. 1.—MARCH 27TH, 1899. SPILLWAY AND CROTON RIVER. LOOKING EAST.

FIG. 2.—MAY 27TH, 1897. MAIN DAM MASONRY.

weighs 185 lbs. per cu. ft. The quarry is connected with the dam by a railroad, and the stone is quarried and sent down in large blocks varying in size, ordinarily, from 1 to 3 cu. yds., although the greater limit is not reached commonly. Stones even of larger size have been furnished occasionally, but difficulty in handling them renders such sizes undesirable.

The spawls and small "chunks" are furnished from quarries along the line of the railroad nearer the dam, and are of the country rock, a laminated gneiss.

In laying the stone, care is taken to see that each stone has been thoroughly cleaned and washed with water in summer, and with steam in winter. The stone is bedded in a heavy bed of mortar in which flat spawls have been placed to "make up" to such hollows or deficiencies as may be apparent in the bed of the stone. The stone in question is then raised, and the imprint it has made in the mortar bed is used as a guide to complete the necessary making up; additional mortar is then placed over the new bed and the stone is lowered again into place, care being taken to place it exactly as it was before. It is then shaken down by bars placed successively at different ends of the stone until the mortar underneath is pressed out on all sides, when, if it is apparent that it "floats" freely without touching the spawls or stones below, it is allowed to remain. Should there be any doubt about this, however, it is taken up a second time, or as often as is necessary to insure a thorough and tight bedding, well made up and with the minimum of mortar left in necessary to the result wished.

The spaces around these stones are then carefully filled with mortar into which smaller stones and spawls are hammered, care being always taken that no small stone shall be hammered into place unless there is an ample bed of mortar under it. All old work is thoroughly cleaned with brooms and washed with water before fresh work is built upon it. It is also carefully sounded with iron rods to make sure that no small stones or spawls have been loosened in the bed. In cold weather the precautions necessary in building on old work are greater, as mortar more or less frozen and disintegrated is commonly found on the surface and the depth of spawls liable to be loosened is much greater.

The general character and appearance of the masonry in the racks is shown in Fig. 2, Plate XLVI. The stones were of such size that an average rise of 3 ft. in the courses was readily maintained. On the

down-stream side the batter called for by the theoretical sections was obtained by stepping. For this purpose selected stones were of course necessary. The quality and appearance of this work are shown in Fig. 1, Plate XLV. The steps were laid out with rises of from 24 to 30 ins., the latter limit preponderating, and the whole of the step is built outside the neat batter line.

On the up-stream face all joints were raked out 2 ins. in depth and were then pointed up with Portland cement, mixed 1 to 1. This includes also the core-wall and spillway masonry as well as the foundations of the main dam. Fig. 2, Plate XLV, shows a section of the up-stream face of the main dam, in which the joints have been raked out and are ready for pointing.

The foundation masonry laid to date in the spillway was laid in 1895, since which time nothing further has been done. It is shown in Fig. 1, Plate XLVI. The masonry work of the lower part of the core-wall was begun early in the history of the dam work at the extreme south end, and was so prosecuted that by the time the masonry of the main dam foundation had reached the point of junction with the core-wall there were only 100 ft. of wall foundation to be done to complete the connection. This has since been done, and the wall is being built up to the surface of the trench excavated for it.

The refilling against the main dam foundations calls for no special comment except in one instance where, on the up-stream side between Station 6 + 12.5 and Station 6 + 62.5, the bad rock forming the face of the excavation at this point continued to fall into the pit, breaking back of the original excavation lines, after the excavation had reached the bottom, and while the up-stream face of the masonry was building. By the time this masonry had reached a height sufficient to be out of danger from the gradually falling rock, a large mass of the latter had fallen in behind the wall at the bottom into the space which during this time had been used as a sump. It was impracticable to get this material out, as the overhanging rock made it too dangerous for men to attempt it without very thorough protection from above, and it was at the same time extremely desirable to have at this point and elevation especially compact back-filling, rather than a large quantity of loose rock.

The plan and sections in Fig. 10 show the extent to which the rock broke back of the lines and the amount and extent of the broken rock

or debris which fell in from above while the masonry wall was being carried up.

This space was used as a sump-hole, the suction pipe being kept at a low elevation at about Station 6 + 20, at which point the rock slope stood. The water flow through the face of the rock was very free, as the face was full of open fissures, particularly through that part of the face shown in the plan as furthest to the right. This formed quite a recess, and through it came at this time most of the flow pumped from the up-stream side of the dam.

This water flow brought a large amount of fine silt with it, and it was reasonable to suppose that the debris at the bottom and along the wall was, in the course of time, filled with it. It was found impossible to force pipes through the debris to the bottom to test this, and two holes were therefore drilled through the masonry at such positions and angles as to reach low points in the filling. These holes are shown on the plan and sections. Drilling them was a matter of considerable difficulty, but they were finally forced through, but filled immediately with fine silt forced in from below, and no grout could be pumped into them. Two other holes were started in this vicinity, for the same purpose, but could not be carried through.

In making further attempts to prove the compactness or otherwise of this mass of debris the surface shown in the plan to the left of the dotted line was covered with several feet of fine sand and gravel after the pipes indicated had been forced into the mass as far as possible, which in some cases was 5 or 6 ft.

The inflowing water, as stated above, showed itself particularly in the space to the right of the dotted line, in which no gravel was placed. The pump suction back of the wall at about Station 6 + 00 kept the water well below the general surface of the debris which had been covered with gravel, and into three of the eleven pipes 17½ bags of American cement (1 to 1 mixture) were pumped as shown. The other pipes, which were tried later, would take nothing, and the job was finally completed by pouring a large amount of grout, 30 bags of American cement (1 to 1 mixture), into the water space back of the dotted line. This pouring was kept up until the water flow was stopped at the point under treatment, and was forced through other seams in the rock face more directly to the sump-hole which was backfilled later with gravel and sand in the ordinary way.

The Pumping for the Main Dam Foundation.

The work of pumping began in April, 1895, a 10-in. two-cylinder Worthington pump being installed at first. It was placed as near the sump as possible, and was fed from boilers placed at the top of the slope on the down-stream side, not far away. Two 100 H.-P. boilers were installed at this time. This number was increased later to four in all, and at certain times, when the demand for steam for the pumps was heavy, but little outside use was made of the boilers, although the excess in boiler power was at least one, when they were all working at their full capacity. As a rule, however, the whole four were kept continuously in use, working moderately after the 10-in. pump had been replaced, a few months after its installment, by a 12-in. compound, double-cylinder pump of the same make, to which two others of the same size were added later.

With this force of three large pumps two were kept at work at moderate speed, while the third was held in reserve, and the 10-in. pump kept either as additional to the reserve or at times used in connection with a number of smaller auxiliary pumps which were constantly in use during the excavation work and until the foundation masonry was complete, pumping from various points in the bottom to the main sump. From the beginning of pumping operations until November, 1898, the main pumps were kept on or near the lower or down-stream slope of the main cut, and the sump was maintained near by, either on the natural bottom, or, as happened during one winter, in a large hole left in the bottom masonry at a low elevation near the down-stream toe of the dam.

It was extremely inconvenient at times to have to limit main pumping operations to one point and to be obliged to lift all the water from the auxiliary pumps, in some cases over the low-lying portions of recently laid foundation masonry, but the risks to main steam pipes laid across the dam would have been too great, either before or after the beginning of the mason work, while the water flow was large. The discharge was ordinarily through a system of four pipes 12 ins. in diameter, two of which were laid through the lower wing-dam at a low elevation and two through the river wall at a somewhat higher elevation. By this means a considerable lift was avoided, the top of the wing-dam being in the first case about 20 ft. above the pipe openings.

TABLE No. 1.—MEAN MONTHLY TEMPERATURES OBSERVED, IN DEGREES, FAHRENHEIT.

(1)	(2)	(3)	(4)	(5)	(6)	(7)
	APPROXIMATE LOCATIONS OF OBSERVATIONS.					
Date.	8 + 50 / 195 L. 8 + 10 / 195 L.	6 + 50 / 36 R. 6 + 00 / 30 R. 6 + 30 / 30 R.	7 + 60 / 200 L.	7 + 70 / 30 L. Cave.	7 + 00 / 12 R. 6 + 00 / 70 R. River.	In Channel.
	°	°	°	°	°	°
Feb., 1896.....	44.1	50.2	....	....	....	35.8
Mar., "	42.8	50.3	....	....	....	36.0
Apl., "	45.0	51.0	....	....	....	51.5
May, "	50.0	52.7	....	....	....	....
June, "	51.5	54.0	....	....	....	....
July, "	56.7	53.3	51.3	....	....	82.0
Aug., "	61.0	52.0	60.0	....	....	72.0
Sept., "	63.0	51.5	65.0	62.0	....	73.0
Oct., "	67.0	53.0	66.0	64.0	....	....
Nov., "	64.7	....	60.0	60.0	....	56.0
Dec., "	56.6	52.5	62.8	55.6	57.6	35.6
Jan., 1897.....	55.0	51.0	57.0	53.0	55.0	34.0
Feb., "	50.0	50.0	50.0	50.0	53.0	....
Mar., "	46.0	48.7	48.7	48.7	52.0	40.3
Apl., "	42.7	48.0	45.3	47.3	51.3	....
May, "	48.0	....	48.0	48.0	51.0	65.0
June, "	64.0	56.0	58.0	54.0	54.0	76.0
July, "	65.0	56.5	64 0	58.5	56.2	76.0
Aug., "	68.7	60.7	64.7	62.0	61.7	76.0
Sept., "	71.2	62.7	71.2	66.2	66.2	74.2
Oct., "	69.6	64.4	69.6	64.8	65.2	64.5
Nov., "	66.5	63.5	67.0	63.0	64.0	47.7
Dec., "	61.6	61.0	63.6	59.2	62.0	40.0
Jan., 1898.....	56.0	57.0	59.5	57.5	59.5	38.7
Feb., "	46.0	52.0	53.0	52.2	54.7	37.2
Mar., "	43.0	52.2	49.0	51.0	54.4	47.6
Apl., "	45.0	50.7	46.2	49.5	51.7	51.0
May, "	49.2	52.0	49.5	50.0	51.2	60.5
June, "	54.4	54.4	55.4	51.6	50.2	74.4
July, "	61.7	59.0	58.7	53.2	51.0	80.0
Aug., "	64.1	60.0	62.0	56.2	53.5	78.2
Sept., "	68.0	62.2	65.0	59.8	54.4	76.2
Oct., "	68.2	57.0	62.5	60.5	54.0	63.5
Nov., "	68.0	55.3	57.0	....	58.0	49.7
Dec., "	66.0	....	66.0	....	58.0	39.0

Valves at the outer ends of the lower pipes secured the work from the possible danger of back flow when the water was high in the river. In December, 1898, one of the main pumps was placed on the up-stream side of the dam wall, which by this time had been carried all the way across the valley and up to a considerable height above the bottom, and two were kept on the down-stream side, steam for the former being carried across the foundation wall. By this time the amount of water to be pumped had decreased materially, as a large

amount of back-filling had been done on both sides of the dam, and the elevations of the sump-holes had been raised.

During the progress of the main dam excavation it was early noticeable that the flow into the sump-holes seemed to come from particular points on the gravel slopes, following the toes of the slopes down, as the depth of excavation increased. This was noticeable on both the up-stream and down-stream sides, and the flows or springs continued to be identified easily as the work progressed and the outline of the foundation masonry was completed, and the flows or springs on the upper or lower side were separated. These flows were confined to the gravel slopes, through which they came freely, and as the back-filling was gradually raised they were forced back up the slopes to the vicinity of the points where they had originally shown themselves.

At the south end of the main cut and along the sides near the end, where the slopes were nearly all hardpan, practically no water was encountered, although there was a considerable area of gravel and boulder slope under the hardpan near the south end of the side slopes on the "quarters." There was, however, a considerable seepage through the lower half of the very high hardpan slope at the south end which, particularly in winter, through the frost and thaws, caused a good deal of gradual sloughing off of the bank, although at no time was the amount of seepage enough to cause a definite flow from the slope.

A long series of observations of the temperatures, Table No. 1, taken at the points where the flows were best defined, is of interest as indicating, perhaps, some differences in the causes and origins of the various flows observed.

In Table No. 1 are shown six series of observations, including one in the river channel. In Column No. 2 the observations were of a flow on the down-stream side of the main cut, at the point nearest to the up-stream toe of the lower wing-dam. In Column No. 3 the flow observed was near the point on the up-stream side where the heavy flow or spring at Station 5 + 95 was in time developed, and it is assumed that this flow was the same, practically, that showed in the spring when it was reached. In Column No. 4 the observations are of a flow developed on the down-stream slope some distance from the flow in Column No. 2. In Column No. 5 the flow was from the large cave at Station 7 + 70 ± on the up-stream side. The flow of Column

No. 6 was also on the up-stream side, and the observations were taken at first in a large well-hole which was built, temporarily, near the up-stream face of the masonry, and later from the flow which showed outside the masonry line after the well was filled up and the water forced outside of the masonry limits.

The stations at the heads of the various columns show the approximate locations of the points of flow at which the temperatures were taken. In some cases there were variations of location at intervals owing to the shifting of the springs from various causes, such as a deepening of the excavations in the vicinity, or, as in the case of the well mentioned in Column No. 6, some change in the masonry and the channels left temporarily in it. Column No. 7 shows the temperature of the water in the river.

An examination of these observations shows clearly a certain uniformity in the flows from the excavation, particularly in regard to the times of extreme temperatures, which occur in September or October and March or April. It is evident that there was no direct connection with the water flowing in the river, and that the two springs observed on the down-stream side correspond closely, as might have been expected, while the other three springs located on the up-stream side are uniform in showing less extremes in temperature and also a close correspondence with each other. In the river the extreme temperatures shown were in January or February and July.

The flow observed in Column No. 3 shows, however, but little variation during the first twelve months, quite in contrast to the others. The observations during these months were evidently of the water from the heavy spring which, when solid rock bottom was reached, was found to flow from the large erosion at Station 5 + 95, 12 R. The point at which the temperatures were taken in this case was some distance from the spring hole as finally defined, and the water was then piped for some months directly to a subsidiary sump.

Temperatures of the water, however, continued to be taken as it flowed from this pipe and later from the pipe used to divert the spring flow to the up-stream side of the masonry. The elevation of the outlet of this pipe was increased gradually as the masonry and the back-filling rose, and the observations were continued until the clay driving had stopped the flow of the spring. The observations from January, 1897, showed variations which correspond with the variations observed

at the other points of flow, although they were not so marked in degree. As this spring was quite different in its characteristics from any of the others coming originally from a much greater depth and with the location of its flow confined to one place during all the time, it is an interesting question as to why, after a year's nearly steady temperature, variations corresponding with the surrounding springs should develop. It may be that after the first twelve months this spring flow was, to a certain extent, exhausted, and its temperature was affected in a greater degree by the increasing proportion of ground-water near its outlet. Its evident decrease in flow, as time passed, may be one argument in support of this assumption.

A daily record of rainfall, river flow and pumping is shown on Plate XLVII. The rainfall and river gauge readings are shown from September 1st, 1895. The pumping record is begun in October, 1895. This shows irregularities for the succeeding eight months, which are partly due to lack of systematic observations and records during that time, and partly to the frequent changes in the location and elevation of the pumps, as it was during these months that the increase in the depth of the sumps was the most marked. The duration of each rainfall is indicated as nearly as possible by the span of the bracket, the height showing the total fall in inches. The flow in the river is indicated by observations taken at a gauge a short distance below the dam location, and the diagram shows the depths of the flow at that point in comparison with the low-water elevation, which is about at gauge reading 1.70. The pump diagram is based on the average daily speed, in strokes per minute, of one 12-in. pump, and all observations within the above time limits have been commuted to this basis, as the only one by which a direct comparison of the pumping work from time to time could be had.

As to the actual amount of water pumped, various tests of the pump capacity were made from time to time. The 10-in. pump was stated by the makers to have a capacity of 1 500 000 galls. per 24 hours, with a maximum rate per minute of 36 * strokes. Each of the 12-in. pumps, at the same maximum rate of speed, had a capacity of

* Double strokes, or one complete revolution, or "cycle," of the pump action. The capacity of each 12-in. pump cylinder for one complete stroke was about 29 galls., the diameter of the piston being 17 ins. and the length of stroke 15 ins. The 12-in. pumps were so designated to agree with the diameter of the discharge outlet.

3"
2"
1"
0"
5'
0'
50
0
RAINFALL INCHES
SEPT.
OCT.
NOV.
DEC.
JAN.
FEB.
1895
RAINFALL
RIVER
PUMPS
SEPT.
OCT.
NOV.
DEC.
JAN.
FEB.
MAR.
1896
RAINFALL
RIVER
PUMPS
NOV.
DEC.
JAN.
FEB.
MAR.
APR.
MAY
1897

PLATE XLVII.
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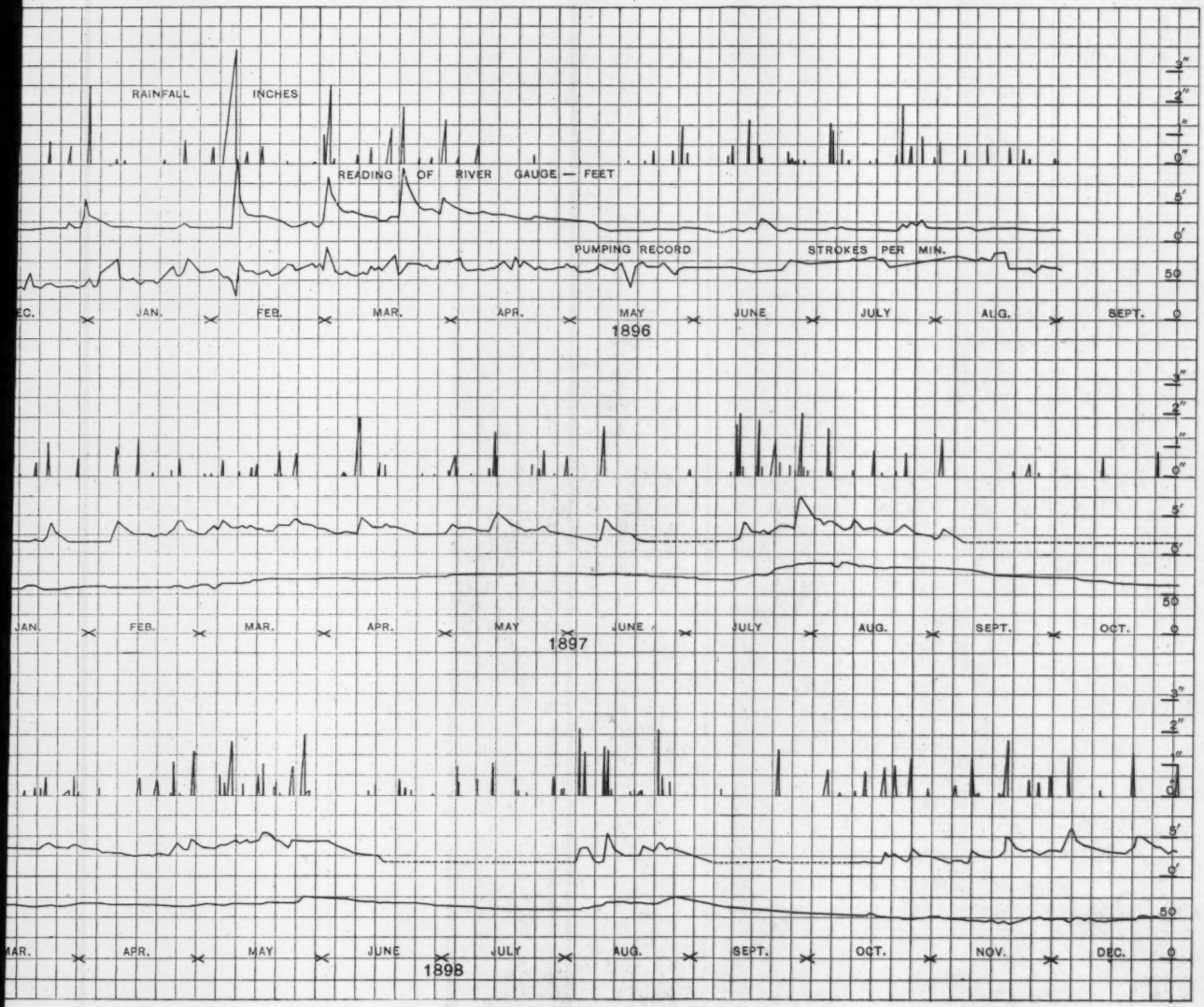
GOWEN ON FOUNDATIONS OF NEW CROTON DAM.



4 000 000 galls. per 24 hours. The tests of the actual amount of water pumped are as follows:

On Sunday, October 27th, 1895, water in the main cut was allowed to rise from Elevation 7.81 to Elevation 10.14, the limits between these elevations having been carefully cross-sectioned. The amount of inflow was estimated as 814 275 galls. in 7 hours, or at the rate of 2 800 000 galls. per 24 hours.

The pumps stopped at 8.40 A. M., and resumed work at 3.40 P. M. The water during this interval rose 2 ft. 4 ins., and seemed to rise at a constant rate of nearly 4 ins. per hour.

* The small Worthington pump (1 500 000 galls.) was unable to control the water in the main cut below Elevation 13. The water gained on the pump up to this elevation, but was then held constant by the small pump.

On December 19th, 1898, an experiment on the pumping capacity of one 12-in Worthington pump was made. The result showed a capacity of 50 galls. per stroke.

The experiment was made by the use of a large sump-hole on the down-stream side of the main dam. One side of this sump was formed by the wall of the dam, the others by the back-filling in progress at the time the experiment was made. The following is a *résumé* of the results. The sump had been carefully cross-sectioned.

DECEMBER 19TH, 1898.

Calculations to determine efficiency of pumps and amount of water flowing into sump. 12-in. pump, down-stream sump.

Capacity of sump between Elevation — 2.2 and Elevation — 8.0	287 512	galls.
Experiment 3.20 P. M. to 7.10 P. M. 230 minutes.		
At 3.20 P. M. the sump was empty, pumps shut off.		
By 7.10 P. M. the sump had been filled by water flowing in from springs. In 230 minutes the amount of water flowing in equals capacity of sump	287 512	galls.
Flow per minute	1 250	"

* Note taken at time above experiment was made.

Experiment 10.30 A. M. to 2.27½ P. M.

237½ minutes.

Sump full at beginning, empty at end.

950 815 Gauge reading at 2.27½ P. M.

939 181 " " 10.30 A. M.

11 634 Number of strokes of pump during experiment.

49 Number of strokes of pump per minute.

Flow during experiment

= 1 250 galls. × 237½ = 296 875 galls.

Capacity of sump	287 512	"
Amount pumped	584 387	"
Divide by number of strokes (11 634) and we get	50.23	" per stroke.

Experiment 2.27½ P. M. to 3.20 P. M.

52½ minutes.

Sump empty at beginning and end.

952 453 Gauge reading at 3.20 P. M.

950 815 " " 2.27½ P. M.

1 638 Number of strokes of pump during experiment. Multiply by 50.23 galls. per stroke and we get

82 277 galls. pumped out during experiment equals gallons flowing in

82 277 ÷ 52.5 = 1 567 galls. Flow per minute (when sump is empty)

1 638 ÷ 52.5 = 31.2, average number of strokes per minute.

Flow per minute with sump empty during experiment	1 567	galls.
Flow per minute with sump empty at beginning and full at end, or *vice versa*	1 250	"
Difference	317	"
Capacity of pump per stroke from experiment	50.23	"
Capacity of pump per stroke (pump measurement)	58.00 ±	"

N. B.—For 24 hours previous to these experiments the water in the sump was held at Elevation — 5.54, and the speed of the pump averaged 29.5 strokes per minute. The rise and fall of the water in the sump in the above experiments was between Elevations — 2.2 and — 8.0.

At 49 strokes (double) per minute the amount pumped is at the rate of 3 550 000 galls. per 24 hours. As the maximum number of strokes shown on the pump diagram is not more than 90 strokes per minute, it may be assumed that the maximum flow into the pit was less than 7 000 000 galls. per day. 29.5 strokes per minute equals about 2 160 000 per day.

A special experiment was made at a time when one 12-in. pump was at work on each side of the dam, in order to detect if possible any variation in the pumping rate owing to the relative difference in the elevations of the sump levels. The result is as follows:

TABLE No. 2.—Calculations to Determine Effect of Elevation of Water in Sumps on Up-Stream and Down-Stream Sides of Dam upon Amount of Water Pumped.

Experiment, December 21st, 1898.

Elevation of water in down-stream sump — 6.55.
" " " up-stream sump + 8.7.
12-in. pump used in each sump.

Time.	Down-Stream Sump.			Up-Stream Sump.
	Register.	Difference.	Strokes per minute.	Strokes per minute.
7 A. M.	017 256			
8 "	018 968	1 712	28½	21
9 "	020 726	1 758	29⅓	19½
10 "	022 415	1 689	28	20½
11 "	024 457	2 042	34	20
12 M.	025 883	1 426	23¾	21
1 P. M.	027 601	1 718	28⅔	21½
2 "	029 306	1 705	28½	21½
3 "	031 030	1 724	28¾	21½
4 "	032 739	1 709	28½	21½
5 "	034 462	1 723	28¾	21½
6 "				21½
Total strokes			286½	231
Average strokes per minute during day			28.6	21.0

Water in down-stream sump lowered during night.

EXPERIMENT, DECEMBER 22D, 1898.

Elevation of water in down-stream sump — 8.01.
" " " up-stream sump + 8.7.
12-in. pump used in each sump.

Time.	DOWN-STREAM SUMP.			UP-STREAM SUMP.
	Register.	Difference.	Strokes per minute.	Strokes per minute.
8 A. M.	061 265			19½
9 "	062 908	1 643	27½	19½
10 "	064 589	1 681	28	19½
11 "	066 252	1 663	27⅝	19½
12 M.	067 893	1 641	27⅓	20
1 P. M.	069 535	1 642	27⅓	19
2 "	071 167	1 632	27⅙	19
3 "	072 807	1 640	27⅓	20½
4 "	074 425	1 618	27	19½
5 "	076 046	1 621	27	20
6 "				21
Total strokes			246½	217
Average strokes per minute during day			27.4	19.7

CONCLUSION.—The relative heights of water on the up-stream and down-stream sides of the dam seem to have no effect on the relative amounts of water pumped.

NOTE.—On December 20th about 0.35 in. of rain fell, which may account for the greater amount of pumping on the 21st than on the 22d.

It is not assumed that the foregoing tests are accurate. They are sufficiently reliable, however, to enable it to be said that the maximum daily pumping did not at any time exceed 7 000 000 galls., and the pump diagram furnishes a reliable comparison between the amount done from time to time and the maximum. These diagrams, it may be said, are chiefly of interest in that they serve to show the relations existing between the rainfall and the resulting river flow and necessary pumping. As to whether any deductions of value can be made, excepting that in a gravel bottom, below sea level, in close proximity to a river large in times of heavy flow, the amount of water pumped was under 7 000 000 galls. per day, while the area of the pump well was at least 3 acres, and the depth 130 ft., remains to be seen.

Plate XLVIII shows curves deduced by averaging per month the various data shown in Plate XLVII. On it are also shown the curve of increasing depth of sumps from which the pumping was done, and a curve showing the increase in yielding volume as the depth of the sumps and the size of the excavation increased. These curves are not extended beyond March, 1898, as by that time the refilling had been

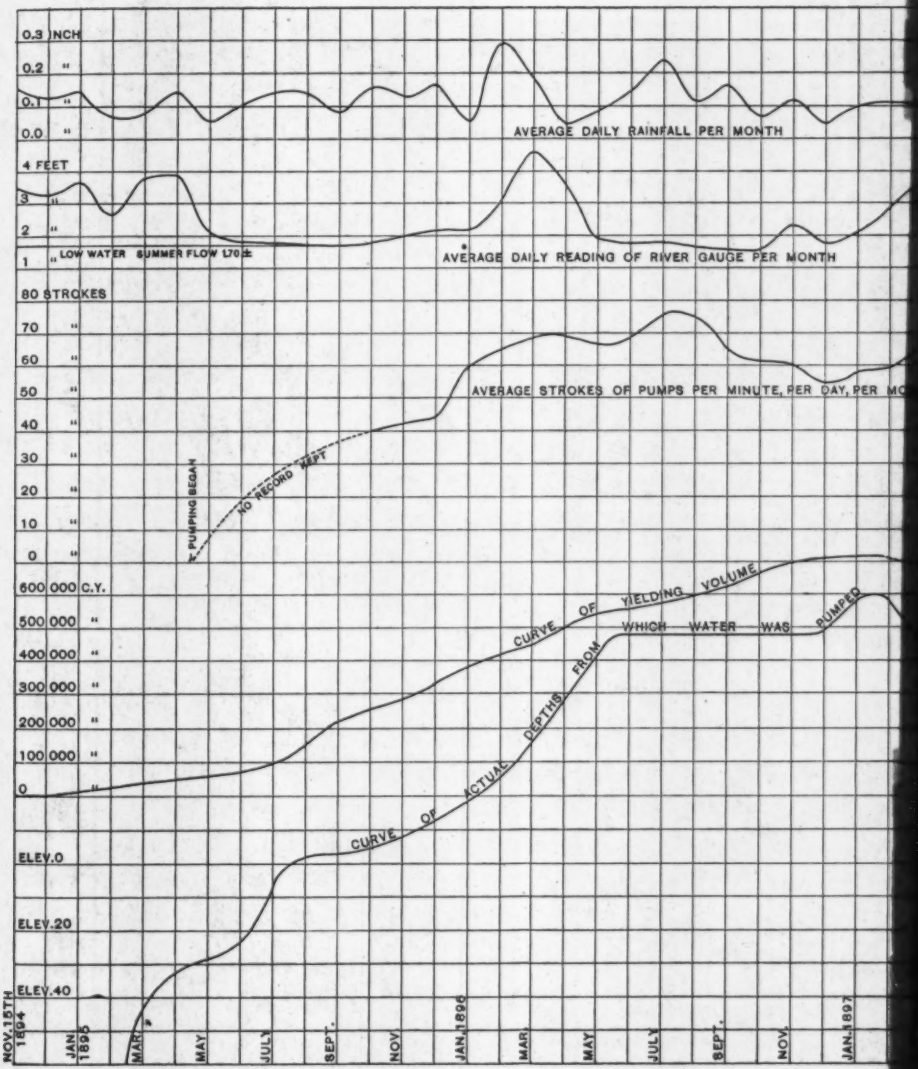

0.3 INCH
0.2 "
0.1 "
0.0 "
AVERAGE DAILY RAINFALL PER MONTH
4 FEET
3 "
2 "
1 " LOW WATER SUMMER FLOW 1.70±
AVERAGE DAILY READING OF RIVER GAUGE PER MONTH
80 STROKES
70 "
60 "
50 "
40 "
30 "
20 "
10 "
0 "
AVERAGE STROKES OF PUMPS PER MINUTE, PER DAY, PER MO
PUMPING BEGAN
NO RECORD SEPT
600 000 C.Y.
500 000 "
400 000 "
300 000 "
200 000 "
100 000 "
0 "
CURVE OF YIELDING VOLUME
WHICH WATER WAS PUMPED
FROM
CURVE OF ACTUAL DEPTHS
ELEV.0
ELEV.20
ELEV.40
NOV.15TH 1894
JAN. 1895
MAR.
MAY
JULY
SEPT.
NOV.
JAN.1896
MAR.
MAY
JULY
SEPT.
NOV.
JAN.1897

PLATE XLVIII.
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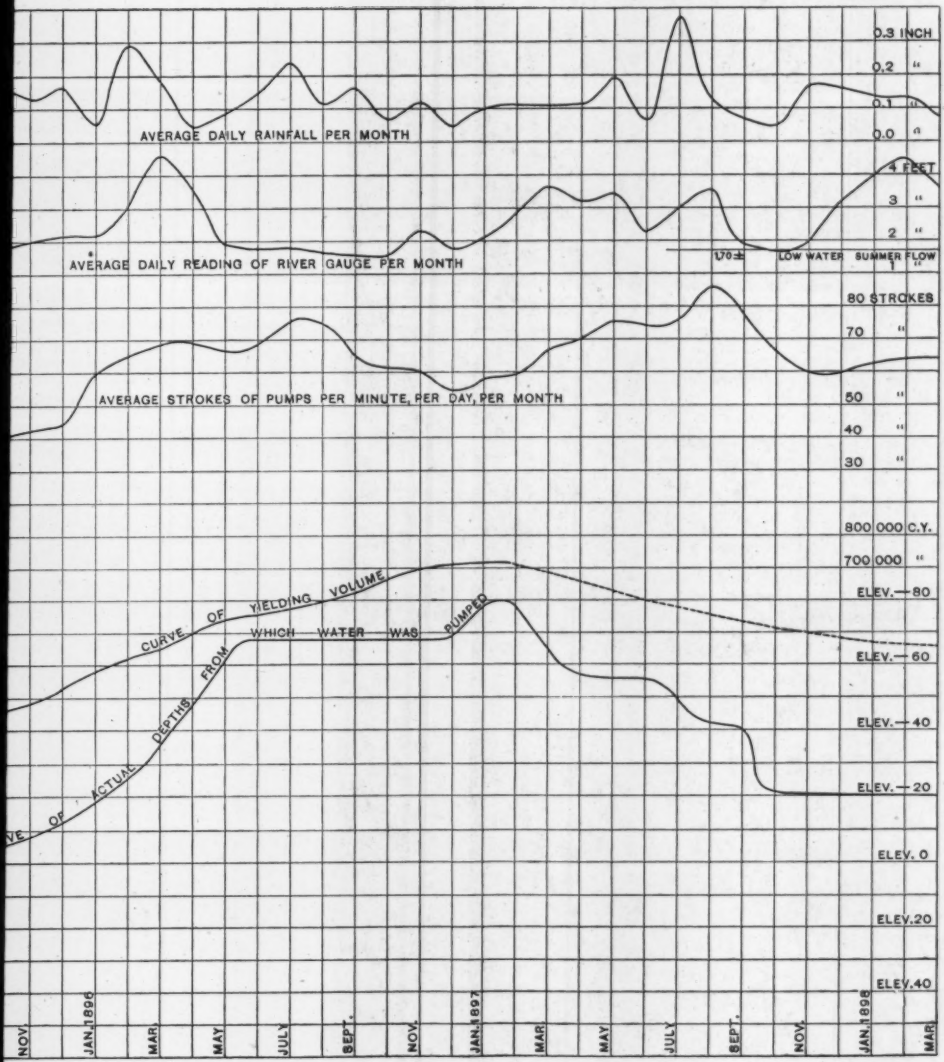
GOWEN ON FOUNDATIONS OF NEW CROTON DAM.



well started, while the whole foundation had been covered with masonry which had been carried up to a considerable height above the bottom.

The curves show, as might have been expected, the effect of the rainfall upon the river which for most of the time is closely corresponding, although the river may be somewhat slower in its action. This does not hold good for the dry summer and autumn months of 1895 and 1896, but the correspondence is close in 1897 when extreme low water was not reached until September. The comparatively heavy flow in February, 1898, however, does not seem to be accounted for by any special rains at about that time.

The pump curve shows a constant rise in 1895 and to May, 1896, as the depth and yielding volume increased. The apparently extra rise in this curve from January to April, 1896, may have been influenced by the rainfall, to which it seems to have responded more quickly than the river. Again, in July and August, with constant depth, there is shown a quick response to the rainfall which is not noticed in the river. In December the pumping had fallen off materially, although the depth and yielding volume were on the increase. The gradually diminishing rainfall, from September to December, may possibly account for this. The steady increase in the rain from January to July, 1897, is noticeable in the river curve and marks a constant increase in the amount of pumping, although the maximum of yielding volume and depth is reached six months earlier. The falling off in pumping from July to December, corresponds fairly, although, perhaps, a month behind in time, with the rain and the river curves, but results from December to March, 1898, are due partly to the influence of the back-filling, which must have begun to make itself felt, and partly, doubtless, to the fact that the yielding volume was beginning, as it did in the previous year, to show signs of being pumped out at the end of the dry season.

It seems fairly conclusive that the flow in the river had, on the whole, but little direct influence on the pumping. In other words, the wing-dams were efficient in stopping anything like a direct leakage or flow from the river to the excavation pit. Another conclusion is that a considerable time elapsed between the rainfall and its effect on the pumps, amounting in certain cases to as much as two months.

The data from which the curve of yielding volume is obtained are due to calculations which show approximately at the end of each two months the amount of sand and gravel on the slopes of the pit which furnishes water storage space. It was assumed that all space in the slopes above an angle of 20° with the horizontal, the vertex being taken at the lowest point in the section excavated and kept clear by the pumps, would yield water, except those parts of the slopes which were formed of hardpan and which were not included in the calculations of volumes.

Fig. 12 shows two diagrams of pumping commuted to equivalent strokes of a 12-in. pump, covering the time from November, 1898, to May, 1899. One of these is the record of the pumping from the up-stream side of the main dam, which was done wholly by a 12-in. pump during that time.

On the down-stream side other pumps were used, as noted on the diagram, but some careful experiments were made by which the relative amount of pump work done has been fairly commuted to the 12-in. standard. The accompanying diagrams, which show in both cases the mean elevations of the two sumps from time to time and at the same time the difference in elevation of the water on the two sides of the dam, certainly do not indicate any connection between the two sump-holes. The up-stream pump shows a very slight increase in March and a very gradual decrease to May 15th, with a gradual rise in water elevation to the middle of February, and no material change later. On the other hand, the down-stream pumping shows a material diminution about February 1st, with many changes of water surface and a very material rise in the sump elevation at about the same time.

The relative elevations of these sumps varied during these months decidedly, the up-stream sump changing from about 17 ft. above the sump on the lower side, to an extreme of about 10 ft. below, most of the change, however, taking place on the down-stream side, as shown. These records are suggestive, in view of the general character of the limestone foundation and the possibility of the presence of open seams and channels below those treated in preparing the bottom for the masonry. With a head of nearly 20 ft., long sustained on the up-stream side, then varied gradually until the head on the other side was 10 ft., there seems to have been nothing, in

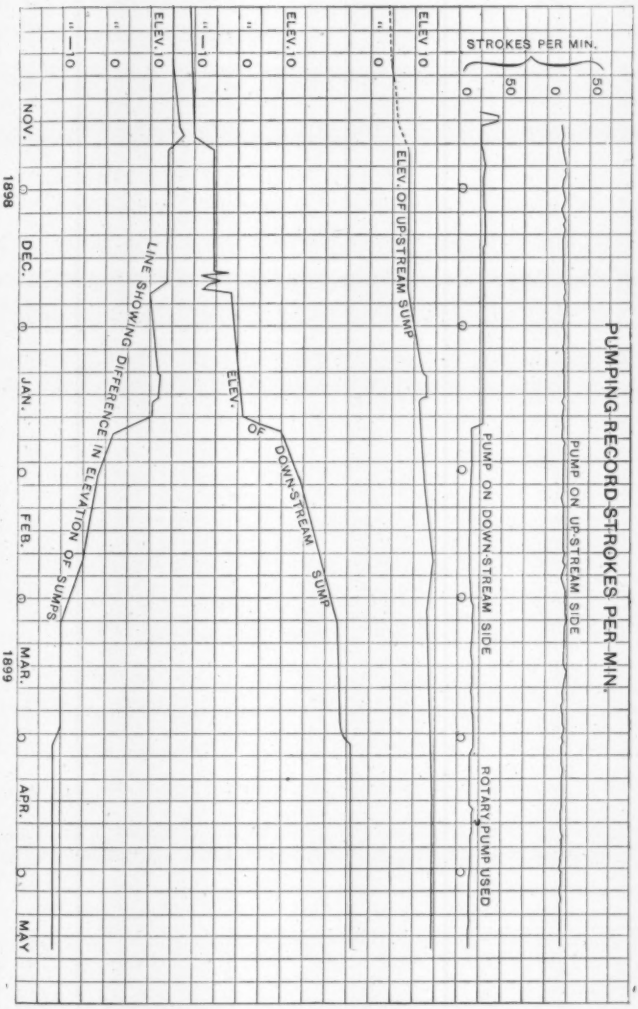

PUMPING RECORD STROKES PER MIN.
STROKES PER MIN.
50
0
50
0
PUMP ON UP-STREAM SIDE
PUMP ON DOWN-STREAM SIDE
ROTARY PUMP USED
ELEV. OF UP-STREAM SUMP
ELEV. OF DOWN-STREAM SUMP
LINE SHOWING DIFFERENCE IN ELEVATION OF SUMPS
ELEV. 10
" 0
" —10
NOV.
DEC.
JAN.
FEB.
MAR.
APR.
MAY
1898
1899

FIG. 12.

this reversal of relative conditions, to affect the flow of water on either side.

General Remarks.

While the purpose of this paper is to deal particularly with all the various features of this work which pertain directly or indirectly to the foundations of the dam, dwelling in some cases in considerable detail upon certain points, it is evident that much information, some of it of interest if not of considerable importance, relating to the general design, construction and progress of the work, has been necessarily omitted. In fact, anything approaching a comprehensive account of these matters would require a book of ample size to give the subject adequate treatment. However, a few figures are added here relating to certain prominent features which have not been noted or described previously.

The main feature of the construction work is the rubble masonry. The total amount required will be not far from 650 000 cu. yds., and of this, at the date of writing, about 405 000 cu. yds. have been laid. The contract price for this item is $4.05 per cubic yard when American cement mortar is used. This price is increased to $4.94 and $5.35 per cubic yard, as Portland-cement mortar (3 to 1) or (2 to 1), respectively, is used. The facing stone masonry, of which it is expected that at least 24 000 cu. yds. will be used, forms another important item. This is used for both the up-stream and down-stream faces of the main dam and overflow above the lines or elevations to which the refilling and embankment will be carried. This facing stone is cut in courses which vary in rise from 15 to 30 ins., having a uniform depth of bed and build of not less than 28 ins. Headers, of which every third stone in each course is one, are not less than 4 ft. in length, and are used to insure a bond with the rubble backing or hearting.

All joints of this stone are cut to lay to $\frac{1}{2}$ in. in width from the face back for 4 ins. in depth on the sides and beds. For the remaining depth the stones must be cut full, to joints not exceeding 2 ins. in width between adjoining stones when laid. In this way there is insured a moderately fine outer joint which is thoroughly raked and pointed to a depth of 2 ins. or more, while the wider 2-in. joints give an opportunity for any settlement that may possibly occur in the future due to inequalities between the relative composition of the facing as compared with the backing stone to which it is bonded.

On this facing stone it is depended to insure the practical water-tightness of such parts of the structure as are exposed directly to water pressure. This stone is laid in Portland-cement mortar, (mixed 2 to 1), and in the pointing of the ½-in. face joint Portland cement is also used. As has been stated previously, on all parts of the up-stream face of the masonry, which are planned to be below the back-filling line, and which are formed of rubble masonry, care has been taken to secure well-shaped stones and to fill up the intervening joints very thoroughly with small stones or spawls. This is well shown in Fig. 2, Plate XLV, where the joints are shown raked out and ready for the pointing which, as in case of the facing stone, is done with Portland cement. In this connection, particular attention may be called to the very great amount of refilling which is to be done on the up-stream side, particularly back of the main dam and above the limestone foundation where so much badly fissured rock was found, and which resulted, necessarily, in the great depth of the rock excavation. This refilling, together with the pointing of the up-stream face of the masonry which it covers, is expected to be effectual in stopping percolation through or under the dam, even if in the latter case small open fissures may exist. At any rate, if, as noted previously in the chapter on "Pumping," a head varying from 20 ft. on the up-stream side to 10 ft. on the down-stream side caused no appreciable variation in the pumping of an amount of water, which at that time might have equalled 3 500 000 galls. per day, it does not seem that the head due finally to a full basin can increase very materially such flow as may possibly have already taken place through the limestone foundation rock.

The contract price for the rock excavation is $1.95 per cubic yard. The amount excavated will slightly exceed 300 000 cu. yds. While the price is seemingly a liberal one, it must not be forgotten that, in the bottom work, the blasting and excavation were not done to ordered lines and grades excepting as so directed from day to day, as the only limit in depth was good rock when reached. This necessitated a very great amount of careful hand work, as well as slow and expensive work in finally getting the bottom ready for the masonry.

All the earth excavation work, which has amounted to nearly 1 100 000 cu. yds., was, under the specifications, let at one price, viz., $0.61 per cubic yard, to avoid complication, although, naturally, there would have been little difficulty in separating the amounts lying below

river level, and involving pumping, from the portion remaining. The price was considered fairly low when the risks were taken into consideration. The increase in the length of the main dam, which was determined upon in 1897, resulted in a considerable decrease in the maximum height of the embankment at its point of junction with the main dam. This change has also decreased the amount of embankment as originally planned, and the core-wall trench is now practically wholly in hardpan; the point of junction with the main dam having been advanced to the south until it found the hardpan overlying the bed rock.

The work of construction began in October, 1892, the contract having been let in the previous August. The time limit in the contract was seven years, bringing the date of completion to August, 1899. The extraordinary depths to which the rock excavation had to be made, in order to secure a foundation for the main dam, as well as the change made in the length of the main dam after the work was started, which involved quite an increase in the amount of masonry, justified an extension of time of perhaps one year. The dam, however, will hardly be finished before 1902, making the time of construction ten years instead of eight. This delay is largely due to the dilatory ways and methods adopted in the first two or three years of the construction work, as developments have shown clearly that the plans and methods proposed by the engineers for carrying on the work have so far provided for and anticipated all emergencies and contingencies, in kind if not in degree; nothing unforeseen having happened to materially delay or involve a change of plans beyond the increased depth of rock excavation found necessary and the increase in the length of the masonry dam, as previously mentioned.

DISCUSSION.

E. SHERMAN GOULD, M. Am. Soc. C. E.—Apart from its intrinsic interest, this paper possesses that of timeliness, in that its presentation coincides so nearly with the retirement of Mr. Fteley from the Chief Engineership of the Aqueduct Commission. It describes the progress of what we need not hesitate to style the most important engineering work of the day, from its commencement up to the date of its handing over, fully and successfully launched, by Mr. Fteley to his successor. Mr. Gould.

This very apt connection between the presentation of the paper and Mr. Fteley's retirement will probably strike all members of the Society. To the more limited circle, composed of members and ex-members of the Corps of Engineers of the Old Croton Aqueduct, the death of Julius W. Adams, Past-President Am. Soc. C. E., occurring so near the date of its presentation, lends to this paper an added appropriateness of time and place.

The preliminary studies for the original project, of which Mr. Benjamin S. Church was the author, namely, that of a dam at Quaker Bridge, and of which the present work will be the final outcome, were commenced some twenty years ago under the direction of the late Isaac Newton, M. Am. Soc. C. E., then Chief Engineer of the Croton Aqueduct, by the late E. S. Chesborough, M. Am. Soc. C. E., and the late Colonel Adams. These studies mark the early dawn of the era of scientific high-masonry dam design in this country. The design and construction of what were previously considered high dams, of earth with a center wall of masonry, had already been brought to a high degree of perfection by Tracy, Campbell and Mr. George W. Birdsall, but a masonry dam, upwards of 100 ft. high, was essentially a new proposition. It was new and startling to many of us at that time to learn that the possible crushing of such a structure under its own weight alone, or under its own weight combined with hydrostatic pressure, was a factor of the problem most seriously to be reckoned with. It is probable that this point, and, indeed, the whole question of the profile of equal resistance of such structures, was first brought to the notice of the profession at large through a translation, made by the writer, by direction of Mr. Isaac Newton, of some chapters from Debauve's "Manuel de l'Ingénieur," of which a small edition was printed by the Department of Public Works for the use of its engineer corps, and copies of which found their way to a few other hands. Mention may also be made of two papers contributed about this time by the writer to *Van Nostrand's Engineering Magazine*. Later, the subject was fully developed by Edward Wegmann, M. Am. Soc. C. E., in his masterly and thoroughly exhaustive treatise on "High Masonry Dams."

Mr. Gould. In these studies, the type or concrete idea of the high masonry dam, which with true engineering instinct was seized upon and kept constantly in view, as a safe precedent, by the consulting engineers, was the dam across the Furens, at St. Etienne, France.

But all these particulars are now ancient history, and appeal only to the very limited number of original pioneers in the study of high dams. Their only general interest lies in the evidence which they may afford of the labor and research which characterized the earliest beginnings of the project now being carried out. It is doubtful if any engineering project in this country has ever been made the subject of so much laborious and painstaking study as that described. This paper, taken in connection with the reports of the Chief Engineer, already published, shows us that these studies, taken up by Mr. Fteley where they were left off by the original projectors, were continued by him with unabated zeal and thoroughness.

The description of the system of borings and other explorations given by Mr. Gowen is noteworthy. The juxtaposition of gneiss and limestone, with outcrops on opposite sides of the valley, seems to be characteristic of this and the neighboring territory, and merits study by the geologist. The disappearance and in some cases reappearance of water in the bore holes at great depths is certainly puzzling to account for. Mr. Gowen calls attention to the discrepancy frequently found to exist between the character of the rock as revealed by the actual excavations and that previously predicted from the borings. This discrepancy was also noticeable in driving the tunnels of the New Croton Aqueduct, and it admonishes us that while the diamond drill is of great utility in preliminary explorations, its indications should be taken with considerable reserve, and interpreted very cautiously.

The quotation from Mr. Fteley's Report (pages 483 and 484), in which he recommends abandoning the Old Quaker Bridge location and building a much smaller dam higher up the stream, is an excellent example of sound engineering judgment, and one is rather surprised at the haste with which, in the face of this recommendation, the Commission adopted the Cornell project. The fourth reason advanced by Mr. Fteley for his recommendation seems, however, to require some modification. He says:

"The interest of the money thus saved for the present would, after twenty-five years, represent a large part of the money necessary to then build the higher dam, with the result that the city would then have two dams instead of one for nearly the same expenditure."

This result could only be safely predicated if the amount of interest saved were year by year paid into a sinking fund, and kept intact. This, it is hardly necessary to remark, would be very unlikely to be carried out.

In this connection, a glance at the estimated amount of storage,

consequent upon the completion of the present dam, will be interesting. On page 471 this amount is stated to be 73 236 000 000 galls. The writer has found that a very serviceable formula representing the relation between the total storage required to maintain a desired daily average consumption throughout the year, and the daily average yield of the source of supply, is: Mr. Gould.

$$S = \frac{C^2}{Y} \times 365.$$

In this equation, S = total storage required; C = daily consumption, and Y = daily average yield of the water-shed, all in the same unit. Let us apply this formula to the present case. The capacity of the New Croton Aqueduct is about 300 000 000 galls. per 24 hours, and this may be taken as the maximum daily consumption of New York, furnishable by this supply. The daily average yield of the Croton watershed above the Cornell Dam, will be about 365 000 000 galls. per 24 hours, We would have then:

$$S = \frac{90\ 000}{365} \times 365.$$

$$S = 90\ 000\ 000\ 000 \text{ galls.}$$

This would be about 300 days' supply, as against 244 actually provided, indicating a fair agreement between the two figures.

Although this paper is confined to the foundations of the great dam, it cannot be satisfactorily discussed without some reference to the profile of the dam itself, which is shown in Fig. 2.

The feature of the profile which immediately challenges discussion is its form below the level of the original surface of the ground. It will be perceived that the profile of equal, or approximately equal, resistance is continued down to the bed rock, some 120 ft. below the bed of the stream. The effect of this is to spread greatly the footing course, and to increase correspondingly the immense volume of material to be excavated. Was this necessary, to insure the required stability of the structure? The writer has no hesitation in saying that both from theoretical and practical considerations he does not believe it to have been necessary or even advisable. He understands that the theory upon which this profile is based is that the dam is, or may be, subjected to the pressure due to a head of water extending from its top, through the excavation and down to the bottom rock on which it stands. He considers this theory as inadmissible. Its acceptance appears to lead to the untenable conclusion that the deeper the foundation, the greater the hydrostatic stress. We have only to consider what a monstrosity would ensue if the dam were carried down to a very great depth, say 300 ft., below the surface and the profile calculated according to this theory. He considers also, that this endeavor to err, if at all, on the side of safety is, to some extent, self destructive, for it

Mr. Gould. results in piling up a huge mass of back-filling upon the outer toe of the dam, adding this extra weight to the already enormous pressure which it is sustaining. The additional spreading of the foot cannot be necessary to resist rotation about the outer toe, because such rotation would be impossible at that depth, nor can it be needed to prevent crushing. That part of the structure which is deeply buried and closely imprisoned on all sides is quite differently and much more favorably circumstanced to resist crushing than is that part which stands entirely above ground with no opposing resistance to prevent the lateral escape of crushed material. In this connection, the writer would quote what he has said elsewhere, as follows:

"Nor can he agree with those who maintain that the thrust of the water from a full reservoir should be considered as that due to a head extending from the top of the dam to the bottom of the foundations. That portion of the dam which is buried in the earth or rock should, in his opinion, be considered entirely apart from the dam proper, and as subject to an entirely different class of stress. He would consider this portion of the structure as forming, in fact, a part of the geology of the territory, and confine his calculations, as regards the thrust of the water, to the superstructure which, standing in relief above the surface of the surrounding ground receives the pressure of the water on one side, and that of the atmosphere only on the other."

Apart from this feature of the profile, some remarks may be made upon the dimensions given to the upper portion. The calculations which lead to the profile adopted were based upon the well-known empirical formulas for resistance to crushing, sometimes called Debauve's formulas, from the fact that they are given by that author in his "Manuel de l'Ingénieur," though not original with him. It is usual to assume that if the profile, so calculated, satisfies the condition of resistance to crushing, it will of necessity satisfy also that of resistance to overturning. This method of calculation results, as regards the latter condition, in a profile which is very light in the upper portion and very heavy in the lower.

In looking at the profile of a very high masonry dam on paper the eye deceives us, unless we keep the scale in mind. We are accustomed to regard such structures—dams and other retaining walls—in reference to their apparent stability as against overturning bodily around the outer edge. Regarded in this way, the profile always seems to be, and indeed is, skimped at the top and redundantly thick at the bottom. This appearance changes, however, when we reflect upon the character of the stress brought to bear upon the base and all the lower portions of the wall. The tendency of this stress is to pulverize the bottom courses, and it can only be resisted by an extension of base beyond what would be required to secure a sufficient moment of resistance to overthrow. To use a homely figure, the dam may be compared to a sack of meal in danger of bursting, rather than to a rigid body in danger of toppling over.

Mr. Gould.

In designing very high dams, therefore, foreseeing as we do the width of base to which we will be forced in order to meet the increasing crushing stress, we instinctively economize in the upper portions, influenced partly, no doubt, by the fact that if failure is to occur anywhere, it had better be at the top than the bottom, and also to make the unit stresses more uniform throughout. But this economy or this desire to equalize crushing stress may be pushed too far. It seems to the writer that this has been done in the present case, where, in his judgment, the outer face of the dam has been hollowed out too much above Elevation 100.

It is stated in the paper that the dam has a general factor of safety of 2 against rotation. It is understood that in all calculations the water in the reservoir is supposed to stand level with the top of the dam, or at Elevation 210. This is certainly an extreme assumption. The spillway elevation is 196, its length 1 000 ft., and its required capacity rated at 15 000 cu. ft. per second. This would correspond to an elevation of about 199, or say 200 at most, in the reservoir. It is probable, therefore, that the dam above Elevation 100 has a factor of safety of at least 2.5, and a rough calculation seems to show this to be the case. But it must be borne in mind that these calculations assume a purely static pressure, due to absolutely quiescent water. This dam, however, will act as the retaining wall, or breakwater, of an immense and deep lake, with soundings of 140 ft., in direct contact with an almost vertical back. Over this lake violent storms will rage, accompanied by wave action of tremendous dynamic force. Is this part of the dam sufficiently massive to meet the shock of these waves and hurl them back upon themselves? Hugh fields of floating ice may be expected to thump heavily against the wall; is it heavy enough to resist their impact urged on by wind and waves? To say the least, we must admit that the practical factor of safety is reduced to its absolute minimum.

The writer would be in favor of placing an earthen embankment, well rip-rapped at the back of the dam, for a portion of its height. The effects of such a bank would be to diminish the chance of percolation down the back and reduce still further the bugbear of an exaggerated hydrostatic pressure; to diminish considerably the effect of the deep wave action against the dam, and by maintaining a constant counter pressure against the back, to limit the range of pressure when the reservoir is alternately full and empty. It would be especially advisable to cover the entire area of the refilled excavation within the reservoir with a heavy embankment in order to consolidate by its pressure the material used in refilling. Plate XXXV shows that the inner slope of the earthen dam on the south side already covers, or is to cover, a large portion of this refilling, so that only an extension of the bank is necessary in order to carry out the suggestion.

Mr. Gould. From the start, it was recognized that the chief engineering difficulties to be overcome in the building of the great dam were the diversion of the river and the taking out of the foundation pit. There can be no doubt that the plan pursued to divert the stream was the best, and its complete success, as recorded in the paper, is the just reward of an intelligent design skilfully carried out. There is no doubt, too, that the method adopted for taking out the excavation, by means of an open cut with sloping sides, was the best under the circumstances, and more certain of success than any attempt to shore up the sides could have been. Had the superstructure been designed to rest upon a base with vertical or nearly vertical sides, the suggestion made originally might have been revived; namely, to take out two comparatively narrow trenches, one at the up-stream and the other at the down-stream face of the foundation, sustaining the sides by means of shoring, and building up these two faces first. The central core of earth could then be removed between these two walls, and the remaining masonry laid. Even in this case, however, the surer plan of side slopes might have been found preferable. Be that as it may, the work has been accomplished successfully as described, and at the present time, when the critical period has been safely passed and the foundations brought up to surface level, there can be nothing but congratulations to all concerned in carrying through this bold and brilliant feat of engineering to a triumphant issue.

The successful prosecution of the work below ground depended upon the ability to keep the pits dry. Evidently, this fact was realized fully by the engineers, and a powerful pumping plant installed for the purpose. It is not always thus, and many operations involving deep excavations are increased greatly in cost, difficulty and danger by inadequate and badly managed pumping facilities.

The overflow arrangement seems to have been intelligently planned, and from an examination of the plans and descriptions contained in this paper the writer thinks it would be difficult to find a flaw in the general design of the work, as regards the handling of the water before, during and after construction.

An interesting feature of the work is the earthen dam with core wall on the south or limestone side of the stream. The writer has not noticed any explicit statement in the paper as to why the change was made from masonry to earth, but it may be inferred readily that the limestone rock was not considered sufficiently solid to warrant a masonry dam, pure and simple, for the entire length. If this was the case, then sound engineering judgment was shown in changing over to earth. It may be suggested, however, that the masonry dam might have been continued across the valley, leaving its down-stream face exposed, while the back was protected by the earthen bank. This would have involved more masonry, the cost of which would be

partially off-set by saving the outer wing-wall and outer earthen embankment. Mr. Gould.

In describing the masonry core-wall introduced in the earthen bank, Mr. Gowen speaks somewhat apologetically of its massiveness. In the writer's opinion it errs in the other direction, and should have been considerably thicker than shown. Its top should, by all means, be carried to Elevation 210, the same as the crest of the masonry dam. Carrying the top of the earthen embankment to 220, as shown in the drawings, is excellent judgment.

In the concluding paragraphs of the section describing the protective work (page 487), Mr. Gowen also speaks somewhat apologetically of the great cost of this work. No word of apology is necessary to justify this entirely wise expenditure. Parsimony here would have been the falsest economy.

The account of the manner of carrying on the deep excavations and of dealing with springs and fissures of the rock is very valuable. It appears to have been successful, and in any event is not open to criticism. This work could only be judged on the ground, and while actually going on. At present it is sufficient to know that what was attempted was accomplished successfully.

Something is said on page 497 about a possible upward water pressure against the bottom of the foundations of the dam. The writer considers that all apprehension of danger from such action is groundless. When the small proportionate area which in any event could be exposed to this action is taken into consideration; when capillarity and friction are given their due weight, and when it is remembered that at the worst it would be a hydraulic, not a hydrostatic, pressure that could take effect, as there would always be a line of escape below the dam, it will be realized that this danger dwindles down to a negligible value. The writer would class fears of this nature with those which prompt a continuation of the theoretical profile down to the deep-seated rock.

All the foregoing remarks apply to what may be called, distinctively, the engineering features of the work. The constructional details are also very interesting.

Before commencing to lay the masonry in the foundation pit of the main dam, it is stated that the rock bottom was painted with a grout of neat Portland cement, which was allowed to set before commencing to build. The writer would question the wisdom of interposing a film of cement between the rock and the masonry. He would prefer to bed the stones directly upon the sharp, clean rock.

The arrangement of the derricks and the racking of the work as described and shown on Plate XLIV, Fig. 2, were judiciously planned, and calculated to secure rapid, systematic and substantial work. The cable-way does not seem to figure in this part of the work. It is not

Mr. Gould. stated whether the "Portland" mentioned was American or foreign, nor whether the "American" cement was of the Portland or Rosendale type. This leaves us a little in the dark respecting the relative merits of the American mortar, 2 to 1, and the Portland, 3 to 1.

The stone used under the name of "gabbro" is probably a syenite, differing from granite or gneiss in that the quartz is replaced by hornblende.

The precautions taken in bedding the stones, described on page 525, are those necessary to secure good hydraulic masonry. It is probable, however, that as the work progressed and the gangs became broken in to the requirements of the inspection, the beds were properly prepared at once, without the necessity of raising every stone in order to rebed it. Foremen accustomed only to ordinary first-class masonry, such as bridge abutments, are apt to be dismayed at first by the lavish use of mortar required in hydraulic work. They soon realize however, that, even when mortar is thrown in by the shovelful none need be wasted, for the surplus is forced out by the stones as they are laid, and goes to form the bedding of the neighboring ones. Specimens of the work are shown in Plate XLVI, and to a larger scale in Fig. 2, Plate XLV. In the latter figure some hammer-dressing or possibly "plug and feathering" seems to have been used to secure approximately vertical joints and horizontal beds, and in these respects the work is satisfactory. It must be recognized, however, that the stones are badly shaped for substantial work. Unless they are all headers, which would be bad construction, they are nearly all too high in the rise for the length of bed, making them top heavy and rendering it next to impossible to secure a good bond, as is plainly seen in the figure. In a wall of the immense thickness of the main dam it is true that many defects of this sort are of comparatively little moment, for the wall must be considered in the mass, and besides, true Portland cement mortar proportioned 2 to 1 or even 3 to 1 is a tower of strength, and covers many sins. But in the case of light work, such as the center wall of the earth embankment, the stones should be got out in such shapes that each individual piece is in stable equilibrium when laid in the wall with its best bed down. They should admit of being laid readily, with a perfect interlocking bond, every vertical joint being well capped by the stone above it, the bond being maintained, not only on the face, but throughout the entire body of the work. In Fig. 1, Plate XLVI, showing the unfinished end of the spillway, a tendency may be seen to produce a triple wall, that is, to build the two faces first, and fill in between them afterward. Unless the greatest care be taken to prevent it, thin work will almost always be built this way, but the result is a weak combination, and the tendency should be carefully guarded against.

The prices paid for the different classes of work furnish valuable

economic data. The price of rubble seems very low, although the facilities for using large stones and working generally to good advantage, are very great. The prices paid for excavation, both rock and earth, on the other hand, seem high. No price is stated for refilling or embankment. It is noteworthy that no concrete is mentioned. Mr. Gould.

The over-running of the time limit in a work so carefully planned, and in which there were no blunders to correct, nor extensive additions made after the work was commenced, is suggestive at the present time when other gigantic undertakings are contemplated by the city.

GEORGE W. RAFTER, M. Am. Soc. C. E. (by letter).—The portions of this paper relating to the borings for determining the nature of the foundations are especially interesting to the writer, as well as the observation that, in the Croton Valley, wherever the bed-rock appeared at one side, it almost invariably dipped down sharply on the other side to a depth at which it would be impracticable to establish a foundation. Mr. Rafter.

This general condition, or a modification of it, is found frequently in streams issuing from the granitic rock horizons of the Adirondack Mountains of New York. The writer has found it repeatedly in his extended series of examinations of sites for storage dams in that region. As to why this phenomenon repeatedly occurs, the geologists are silent; and thus far the writer has been unable to assign any explanation. Certainly, in view of the large amount of high dam construction now projected in the State of New York, an answer to this question would be useful, if for no other purpose than to tell us in many cases what to avoid. An answer is desirable, further, on the general principle that, with the reason fairly understood, we may hope to find more easily the point of minimum resistance—that is to say, the location on a given stream where the conditions are, on the whole, the most favorable.

As to a solution of this problem, the writer considers that undoubtedly it will come through a better understanding of the laws governing glacial drift and the complex phenomena of surface geology, generally.

As stated, some of the Adirondack streams present a modification of the condition described by Mr. Gowen, namely, the bed-rock frequently shows on one side of the valley, dipping down to a few feet below the bed of the stream and then running off on the other side either horizontally or approximately so. This was the condition at Indian Lake, where a dam 47 ft. high was erected in 1898. So far as the studies have been carried, it is the condition at Boreas and Cheney Ponds, Tumblehead Falls, Conklinville and other points proposed as sites for high dams in the Adirondack region, and of which some of the details may be obtained by reference to the writer's reports on the Upper Hudson storage surveys.

Mr. Rafter. In 1893–96, the writer made a series of studies for high dams on the Genesee River ranging from about 100 ft. to 175 ft. in total height. In the work in the Genesee Valley, in 1893, the rocks dealt with were the rather soft and friable Genesee shales. In general terms, the problem was to find material hard enough to carry securely the superimposed weight and, at the same time, insure water-tightness under the foundations and at the ends.

It was deemed desirable to use the diamond drill extensively. The cores taken out showed that the first 20 to 30 ft. of the rock foundation, while evidently capable of carrying the proposed loads, was defective in that there were many minute seams through which considerable water might be expected to escape when under pressure. In order to gain some idea of just how serious a matter this might be, water pressure was applied to drill holes after the drilling was finished, by the use of a rubber packer, placed at different elevations in the holes. Water was forced below the same by means of a pipe passing through the packer. Space will not be taken to describe in detail the arrangements for accomplishing this, because illustrations of such rubber packers may be found in the catalogues of firms dealing in well supplies. Moreover, the appliances and the results attained have been described by the writer somewhat in detail in his reports on the Genesee River storage, for 1893 and 1894. The object in referring to the matter at all on this occasion is chiefly to complete the literature of the subject in the *Transactions* of this Society. So far as the writer is aware, his methods of using water under pressure for testing the quality of rock foundations, as worked out in 1893, were somewhat in advance of methods used previously.*

To illustrate the methods used and the results obtained in 1893, the following abstracts from the log of the tests, as kept from day to day, are presented:

October 17th, 1893.—Tested drill hole *YY4*, using rubber packer set 50 ft. from top of casing. On starting pump, gauge showed 60 to 70 lbs., and pressure rose gradually to 110 lbs. At this pressure the hole took all the water the pump could deliver. After pumping for an hour with the packer at Elevation 539.5, disconnected and found that water ran slowly from top of pipe, about 5 ft. above surface of ground (Elevation 594.0), thereby showing that a small head had been gained at the sides. Packer was then raised to Elevation 559.5 (top of rock at 565.2), and the pump again connected. With the pump at full capacity, the pressure was only 20 lbs. and no more could be gained however rapidly the pump was run. The clear inference is that between Elevations 539.5 and 559.5 there are seams or fissures which allow water to flow out of the drill hole when under about 20 lbs. pressure.

October 19th, 1893.—Tested drill hole *ZZ1*. Packer was first set 19 ft. above bottom (Elevation 532.5). Pump stalled at 100 lbs.

* Report on Genesee River Storage Surveys, Annual Report of the State Engineer and Surveyor of New York, for 1893, p. 416. Also, same report, 1894, p. 360.

pressure. Raised packer gradually, packing it at every few feet by setting pressure above against pressure of water from below. In this way the hole was tested for its entire length and found to stand, as stated, 100 lbs. at the bottom, and from 40 to 50 lbs. in the upper part. Mr. Rafter.

October 31st, 1893.—Tested horizontal hole at foot of Hog-back. Set packer 87 ft. in, or 8 ft. from end. On starting pump, gauge showed 100 lbs. and rose gradually to 140 lbs. Released packer and stopped every 10 ft. until 54.5 ft. from bottom was reached. At this point gauge dropped to 100 lbs., but in 30 minutes advanced again to 140 lbs., and in 15 minutes more to 150 lbs., where it remained for 30 minutes and then dropped to 60 lbs., the steam pressure remaining the same. In 1 hour and 45 minutes the pressure was 40 lbs. At this point water dripped from the side of the Hogback for some distance to the South.

November 2d, 1893.—Repeated the foregoing test with packer 55 ft. from bottom of hole, and with 4 qts. of wheat bran below packer. Pumped with 100 lbs. pressure for 2 hours without effect.

November 2d, 1893.—Tested hole 23 + 42, W. 250, at Hogback location. Hole 84 ft. deep, 14 ft. to rock (elevation of bottom 503.4, top of rock 571.4). Set packer 8.5 ft. from bottom. Gauge showed 165 lbs. Raised packer to Elevation 524.4, and pressure dropped to 80 lbs. Disconnected and put in 4 qts. of bran, whereupon pressure rose to 170 lbs. Raised packer to Elevation 534.0 and still maintained same pressure. At Elevation 536.0 pressure dropped to 100 lbs. and remained at that point even after the addition of 5 qts. of bran. Pressure finally fell to 60 lbs. and remained there for 3¼ hours. While pumping this hole, water ran from casing at 23 + 42, W. 350.

November 4th, 1893.—Tested 23 + 42, W. 350, at Hogback location. Set packer 10 ft. from bottom (elevation 518.0), and obtained 40 lbs. pressure. Added 4 qts. of bran without effect on the pressure. In 2½ hours the pressure rose gradually to 65 lbs. Coloring matter was added, and showed in the water flowing from casing at hole 23 + 42, W. 250. On stopping pump, it was found that water pumped into hole had acquired a back pressure of 20 lbs. On disconnecting, water ran from pipe for 1 hour and 49 minutes. This test indicates not only a connection between hole 23 + 42, W. 250, and this one, under the river bed, and independent of it; but also shows backing up, probably in vertical seams, at the sides of the gorge. A number of other tests at this site gave the same result. In one of them the back pressure increased gradually to from 50 to 60 lbs., where it remained stationary during 4 hours' continuous pumping. Water was then discovered running from a fissure in the rock side of the gorge over 100 ft. above the river surface and several hundred feet away.

November 24th, 1893.—Tested B. 40 + 70, W. 750, at Site No. 1. Set packer 6 ft. from bottom. Pressure rose to 180 lbs., when pump stalled. Raised packer 10 ft., or to 16 ft. from bottom, when pressure rose at first to 170 lbs., but in a few minutes fell to 120 lbs., where it remained for 10 minutes, and finally fell to 100 lbs. Uncoupled and added bran, when pressure rose from 100 to 120 lbs. Raised packer to 26 ft. from bottom and gauge showed 40 lbs. Again added bran and gauge rose to 50 lbs. With packer at this elevation gas issued from casing at hole B. 40 + 70, W. 850. Upon raising packer 2 ft. more, larger quantities of gas flowed from the hole. The packer was raised and lowered several times with like results, showing a connection between the two holes at about Elevation 548.0.

Mr. Rafter. The use of wheat bran, as referred to in the foregoing, was for the purpose of determining whether or not the seams permitting the escape of water were of minute or open texture. When minute, the fact was shown quickly and easily by the use of a very small quantity of bran.

In 1896 the final studies for the Genesee storage dam at Portage were made. Here very simple conditions prevailed. The rocks dealt with were the comparatively hard sandstones of the Portage group, and a few borings followed by water-pressure tests removed all uncertainty as to the nature of the foundation. The writer has no doubt that diamond-drill cores and a series of water pressure tests properly carried out can be made to yield more for a given expenditure than any other method of investigating this specific problem thus far devised. Indeed, there is really no other satisfactory method of investigation.

In view of the interesting and valuable results obtained, even from observation of loss of drill water at the New Croton Dam, it seems unnecessary to dilate at length on the practical value of such tests.

The method of stopping cavities by forcing in plastic clay is interesting and undoubtedly new to most engineers. Mr. Gowen is fortunate in the considerable number of either new, or substantially new, details developed on the work under his charge, and which he has presented in this paper.

Mr. Le Conte. L. J. Le Conte, M. Am. Soc. C. E. (by letter).—This paper is a valuable contribution to practical knowledge on masonry dam building, particularly where the bed-rock at the site is of inferior character. It contains many interesting data relating to the treacherous nature of limestone formations. Every student of the stability of masonry dams, however, will feel a certain amount of disappointment in the fact that systematic efforts were not made to determine the amount and extent of the expected up-lift on the base of the dam, due to the upward pressure of the ground-water when the lake is filled.

It will be remembered that during the building of the Vyrnwy Dam Mr. Deacon went to much expense and made many useful experiments with the view of determining the amount and extent of up-lift on the base of the dam, and the results were both instructive and thoroughly convincing.

The mammoth dam now being built on the Cornell site, based on a seamy bed-rock full of running water, certainly furnished rare opportunities for making further valuable experiments in the same direction, and it seems strange that some efforts were not made to get more extended information on this all-important subject, and at no great additional expense. It is extremely doubtful how much of this up-lift can be suppressed by grouting wet seams and filling up cavities in the limestone bed-rock. Where hydrostatic pressures are great, as they will

be in this case, it is hard to understand how the up-lift will be confined to the mouths of the bed-rock fissures exclusively. Mr. Le Conte.

The author states that a great many test holes were drilled and piped, the ground-water in some rising 83 ft., with the lake as yet empty.

It is to be hoped that some of these pipes have been, and will be, maintained and continued up vertically through the completed dam, with the view of noting the changes in the pipe water-levels as the lake fills up.

J. L. Power O'Hanly, M. Am. Soc. C. E. (by letter).—The Croton Dam is a lasting monument to the engineering skill of the expiring years of the nineteenth century. It will remind future generations that American engineers of this age and nation could not only conceive, but execute, great and daring projects. Mr. O'Hanly

In venturing on a brief and desultory criticism of the foundations of this stupendous structure, it appears to the writer that the engineer, in his extreme caution in preparing the rock foundations, leant rather too much toward the side of timidity.

The writer can see no good reason to justify any excavation below Elevation — 25. The testimony of Professor Kemp seems conclusive that little danger need be apprehended from subterranean caves, fissures or springs.

There is nothing in the author's diagnosis to indicate any chemical or mineral difference in the ingredients or constituents of the "hard white rock" and the "soft white rock," except the relative quality of hardness. "Soft white rock" may be, and probably is, "hard white rock" in one of its stages of growth or decay. It may be inferred that either all the "soft white rock" will grow, mature and develop into "hard white rock," or that the latter will deteriorate to the state of "soft white rock." In either case the precaution would be futile.

Have any tests been made to determine whether this "soft white rock" has, to any appreciable extent or to any extent at all, been compressed by the maximum pressure caused by the dam? If the rock stood this test the writer would consider it perfectly safe, by analogy, viewed simply and solely as one of the strata, natural or artificial, constituting the geological formation of that locality.

The following are a few of the reasons which have led the writer to these conclusions. Happily for critics, they are absolved of much of the responsibility which appertains to the executive.

Rankine's notation has, from its familiarity, been followed in these formulas and calculations. The masonry is taken at 172 lbs. per cubic foot, and was calculated as follows: The stone weighs 185 lbs. per cubic foot, and the mortar, which comprises about 17% of the mass, weighs 110 lbs. per cubic foot. The "dirt," overlying the up- and down-stream faces of the dam, is taken at 120 lbs. per cubic foot. The dimensions have been obtained chiefly from Fig. 2.

Mr. O'Hanly.

With origin of co-ordinates at O, the extreme point of the masonry of the up-stream face at bed-rock, Elevation — 25, the axes of co-ordinates are Ox and Oy, the former horizontal, the latter vertical. The extreme width of the dam at bed-rock, Elevation — 25, is taken at 206 ft.

S_1, S_2, etc., are masonry sections; S'_1 and S'_2 are sections of the superincumbent earth on the down- and up- stream faces, respectively, of the dam. W is the weight in pounds of a section of the dam 1 ft. long. Wx is the moment in foot-pounds around Ox, and Wy around Oy, and Σ the sum.

TABLE No. 3.—WEIGHTS AND MOMENTS OF DAM.

No. of Section.	Between Elevations.	ΣW.	ΣWx.	ΣWy.
S_1	—25 and 00	819 000	9 959 000	79 771 000
S_2	00 " 20	562 000	19 558 000	49 906 000
S_3	20 " 40	489 000	26 748 000	39 902 000
S_4	40 " 60	411 000	30 702 000	30 167 000
S_5	60 " 80	334 000	31 630 000	21 743 000
S_6	80 " 100	267 000	30 598 000	15 726 000
S_7	100 " 120	205 000	27 573 000	10 947 000
S_8	120 " 140	151 000	23 330 000	7 203 000
S_9	140 " 160	112 000	19 544 000	4 738 000
S_{10}	160 " 180	84 000	16 338 000	3 217 000
S_{11}	180 " 200	67 000	14 372 000	2 345 000
S_{12}	200 " 210	31 000	7 230 000	1 085 000
S'_1	—25 " 68	508 000	31 496 000	89 256 000
S'_2	—25 " 68	89 000	5 518 000	475 000
		4 129 000	294 616 000	356 481 000

$x_O = 71.35.$ $y_O = 86.3.$

These are the co-ordinates of the center of gravity. The center of pressure falls well within the middle third of the foundation, which insures stability of position.

The area of a unit section of the foundation is 206 sq. ft. = 29 664 sq. ins.

The pressure on the foundation, caused by the weight of the dam and the superincumbent earth, is 140 lbs. per square inch.

The crushing stress,* in pounds per square inch, ranges from a maximum of 16 893 lbs. for Grauwacke, 8 528 lbs. for compact limestone (strong) to 3 050 lbs. for magnesian limestone (weak). The pressure exerted by the dam on its foundation is barely one-twenty-second of the crushing stress of this weakest substance bearing the name of rock.

The Pressure of the Water Against the Dam.—The weight of 1 cu. ft. of water = w' = 62.4 lbs., and it is assumed that the crests of waves may occasionally reach the top of the dam. Hence the maxi-

* Rankine's "Manual of Civil Engineering," 7th edition, page 361.

mum height of the column of water, x, is 142 ft. The inclination from the vertical of the imaginary line joining the top of the dam and the bottom of the column of water is $j = 4$ degrees. Mr. O'Hanly.

The pressure of a unit section of the column of water against the dam is

$$P = \frac{w^1 x^2}{2} \sec j = 31.2 \times 20\,164 \times 1.0024419 = 631\,000 \text{ foot-pounds.}$$

Table No. 4 shows the weights and moments of that part of the dam above the bed of the reservoir, and resisting the direct thrust of the water. The masonry, as before, is taken at 172 lbs. per cubic foot. The notation is the same. S'_1 is the column of water resting on and supported by the curved outline of the face of the dam.

TABLE No. 4.

No. of Section.	Between Elevations.	ΣW.	ΣWx.	ΣWy.
S_1	68 and 80	192 000	1 113 000	12 192 000
S_2	80 " 110	267 000	5 767 000	15 726 000
S_3	100 " 120	205 000	8 508 000	10 947 000
S_4	120 " 140	151 000	9 287 000	7 208 000
S_5	140 " 160	112 000	9 128 000	4 738 000
S_6	160 " 180	84 000	8 526 000	3 217 000
S_7	180 " 200	67 000	8 161 000	2 345 000
S_8	200 " 210	31 000	4 247 000	1 085 000
S'_1	68 " 210	44 300	4 194 000	930 000
		1 152 300	58 951 000	58 383 000

$x_o = 51.16.$ $y_o = 50.67.$

These are the co-ordinates of the center of gravity of that portion pressed against by the water.

The condition of stability of friction at any bed joint—the joint at the bottom of the reservoir at Elevation 68—is that the ratio of the horizontal component of the water pressure to the sum of the weight of the dam above that joint and the vertical component of the pressure of the water shall not exceed the tangent of the angle of repose of the masonry, usually taken at about 0.74.

This ratio, in symbols, is

$$\frac{P \cos j}{W + P \sin j}$$

which reduces to

$$\frac{w^1 x^2}{2\ W + w^1 x^2 \tan j} = \frac{1\,286\,300}{2\,304\,600 + 90\,000} = 0.54 < 0.74.$$

Mr. O'Hanly.

Overturning moment:

$$M = \frac{w^1 x^2}{2} \sec j \left\{ \frac{x \sec j}{3} - (q + \tfrac{1}{2})\, t \tan j \right\}$$

$$= \frac{w^1 x^3}{6} \sec^2 j - w^1 x^2 t \left(\frac{q}{2} + \tfrac{1}{4} \right) \tan j$$

$$= 27\,596\,000 \text{ foot-pounds.}$$

The arm of the equivalent couple is $\frac{27\,596\,000}{1\,152\,300} = 24$ ft., which transfers the point of application of the force 24 ft. to the left of the vertical from the center of gravity.

This leaves the center of resistance, F, 59 ft. back from the face, at Elevation 68, or 40 ft. from the toe of the dam, and well within the middle third of the joint.

The Joint at Elevation 140.—The center of gravity of that part of the dam above Elevation 140 is found as shown in Table No. 5:

TABLE No. 5.—WEIGHTS AND MOMENTS.

No. of Section.	Between Elevations.	ΣW.	ΣWx.	ΣWy.
S_1	140 and 160	112 000	1 064 000	4 738 000
S_2	160 " 180	84 000	2 478 000	3 217 000
S_3	180 " 200	67 000	3 337 000	2 345 000
S_4	200 " 210	31 000	2 015 000	1 085 000
		294 000	8 894 000	11 385 000

$x_o = 30.25.$ $y_o = 39.$

These are the co-ordinates of that part of the dam above Elevation 140, the vertical from the center of gravity cutting the joint nearly in the center of the figure.

To Find the Center of Resistance of the Pressure against that Part of the Dam above Elevation 140.—The condition of the stability of friction at the bed joint at Elevation 140 is that the ratio of the horizontal component of the pressure of the water to the sum of the weight of the dam above that joint and the vertical component of the pressure of the water shall not exceed the tangent of the angle of repose of the masonry, as explained already. The ratio is

$$\frac{P \cos j}{W + P \sin j},$$

which reduces to

$$\frac{w^1 x^2}{2\,W + w^1 x^2 \tan j}.$$

But the up-stream face of this part of the dam is vertical. Therefore $\tan j = 0$, and the above expression becomes Mr. O'Hanly.

$$\frac{w^1 x^2}{2 W} = \frac{62.4 \times 4\,900}{2 \times 294\,000} = \frac{306\,000}{588\,000} = 0.52 \leq 0.74.$$

This insures stability of friction at that joint.

Overturning moment:

$$M = \frac{w^1 x^2}{2} \sec j \left\{ \frac{x \sec j}{3} - (q + \frac{1}{2}) t \tan j \right\}$$

With face vertical, $\sec j = 1$, and $\tan j = 0$, which reduces to

$$M = \frac{w^1 x^2}{2} \times \frac{x}{3} = \frac{w^1 x^3}{6} = 10.4 \times 343\,000 = 3\,567\,000 \text{ foot-pounds.}$$

The arm of the equivalent couple is

$$\frac{3\,567\,000}{294\,000} = 12 \text{ ft.}$$

The vertical from the center of gravity intersects the joint 13 ft. from the face. The center of resistance, F, is shifted to the left, or 25 ft. from the face of the dam, being 12 ft. from the toe, slightly outside the middle third of the joint. This seems to be a weak spot in the dam.

On page 248 of Rankine's "Manual of Applied Mechanics,"* occurs the following:

"*Example III.—Triangular Wall with Vertical Axis.*—When the wall stands on a soft foundation, it may be desirable in some cases so to form it, that the center of resistance, F, shall be at the middle of each joint, and shall also be vertically beneath the center of gravity of the part of the wall above the joint. In this case, the point of intersection, A, of the lines of action of the pressure and weight must also fall in the middle of each joint. To fulfil these conditions, the vertical section of the wall should be an isosceles triangle, the outer and inner faces forming equal angles j on opposite sides of the vertical axis of the wall, and the angle j should be such that a straight line perpendicular to $O\ D$ at C shall bisect the base; that is to say,

$$\text{"}\frac{t \sin j}{2} = \frac{x \sec j}{3}$$

"but

$$\text{"}\frac{t}{2x} = \tan j$$

"whence we have

$$\text{"}\sin^2 j = \frac{1}{3}; \cos^2 j = \frac{2}{3}$$

$$\text{"}\tan \frac{t}{2x} = \sqrt{\frac{1}{2}} = 0.707; \text{ and } j = 35\ \tfrac{1}{4}^\circ$$

"so that the base of the wall is to its height as the diagonal to the side of a square."

How is this to be interpreted? Is it to be interpreted literally? That a dam of whatsoever height, on any foundation, is stable with

*Fifth Edition. London, 1870.

Mr. O'Hanly. dimensions of these proportions? Or is it only applicable within the narrow limits of a low dam? Would the Croton dam, executed on these lines, be safe and stable?

It is assumed that to prevent percolation a wall of adequate thickness at the upper face would needs be built down to bed-rock, that the original earth surface be excavated 8 ft. deep, to be overlaid with a bed of concrete 6 ft. in depth, with the dam masonry and wall connected with an impervious layer of concrete.

The base of the dam masonry would be at about Elevation 40, with a height of dam of 170 ft. Therefore, by the rule, the width of the dam at the foundation would be $170 \times 1.414 = 240$ ft.

But if the dam were founded on the "restored surface," the width at the foundation would be 204 ft. Would this be practicable?

The thanks of members are justly due to the author for this valuable and interesting paper. It would add much to its value if the author could conveniently supplement it with additional information, such as the average monthly strength of the skilled and unskilled force, the quantities of materials and cost of the several works mentioned, the current rate of wages, and a detailed statement of the kind of plant in use and its cost.

Mr. Gowen. CHARLES S. GOWEN, M. Am. Soc. C. E. (by letter).—In presenting his paper on "The Foundations of the New Croton Dam," the writer did not anticipate that the question of the section adopted would be raised as the principal point of one of the discussions offered, and he would, for obvious reasons, prefer to leave the further discussion of this point to those who were originally more interested than he in the matter. Nevertheless, the following is offered in reference to this question, in connection with his reply to the other points raised by Mr. Gould.

As the writer understands it, the following were the principal conditions governing the design of the section:

The water level when the basin was full was taken at Elevation 206.

Water pressure was assumed to obtain to the level of the bed-rock surface.

The back pressure due to the water on the down-stream side (water-table level) was also taken into account and allowed for.

Pressures (calculated) were limited to 15 tons per square foot at the base of the structure (rock surface), and the lines of pressure were kept well within the middle third of the section at any assumed level.

The above conditions are stated because Mr. Gould seems to have been in error in understanding that, in the calculations, Elevation 210 was assumed as high-water mark and that the profile of equal resistance was continued down to bed-rock, 120 ft. (instead of 75 ft.) below the bed of the stream; while, as to the elevation of the overflow (196), provision has been made for an increased height by means of flash-

boards at some future time, and a high-water elevation of 206 is not improbable. Mr. Gowen.

No additional thickness of section was made on account of possible ice pressure or wave motion. It is fair to assume that the extensive overflow located in close proximity to the main dam will operate at ordinary high water to relieve the lake of extensive areas of ice nearly as effectively as if it were on the main dam proper, while the width of the structure at the top (Elevation 210), necessary for a roadway, gives additional weight and stability to the section immediately below.

Mr. Gould maintains, apparently, that no account need be taken of pressures below the restored natural surface, below which the foundations may lie, and that no spreading of the "foot" should take place below this level. In other words, that the structure should be designed only with reference to its height above the restored natural surface; that the base on which it rests should be limited by vertical sides; and that dependence should be placed upon the weight of the refill for resistance to crushing and overturning, both of which tendencies would necessarily be increased greatly by the narrow foundation width. In support of this assumption he has stated that "he would consider this portion of the structure as forming, in fact, a part of the geology of the territory," which would seem to mean that the foundation wall must be taken as equal to the ledge rock below, at least in its capacity to resist crushing strains, and that the refilling or restored material must be as compact as it was originally, before excavation was made, in order to be as effective as possible against tendency to overturn. Is not this assumption extreme?

Further on, Mr. Gould states that the tendency of the stress, on the base and lower portions of the wall of a high masonry dam, is to pulverize the bottom courses, and that it can only be resisted by an extension of the base beyond what would be required to secure a sufficient moment of resistance to overthrow. He then compares such a section, perhaps not inaptly, to a bag of meal in danger of bursting. When, however, he proposes to extend such a section for an indefinite distance below the ground surface, and depends upon the refilling to counteract the bursting and overturning tendencies, it would seem to the writer that the plan is somewhat analogous to that of packing the bag of meal in shavings to avoid further chance of rupture to the bag.

It may be of interest to compare results obtained by Mr. Gould's formula for determining the required storage, with results derived by other methods, and if in case of the New York City water supply we assume $C = 280\,000\,000$ galls. per day—which is all that the Croton is calculated to supply in dry years—and $Y = 360\,000\,000$ galls. per day, which may be taken as a fairly conservative estimate of the average

Mr. Gowen. yield of the water-shed, as deduced from observations extending from 1870 to 1894, inclusive, we have as the required storage—

$$S = \frac{280\,000\,000^2}{360\,000\,000} \times 365 = 79\,490\,000\,000 \pm \text{galls.}$$

Deductions made on the basis of the Sudbury River records from 1875 to 1895,* inclusive, would give the following as the required storage for a dry year supply of 280 000 000 galls. per day—

Area of Croton water-shed 361 sq. miles.

$$\frac{280\,000\,000}{361} = 776\,000 \pm \text{galls. per square mile, average yield per day required.}$$

This requires a storage per square mile of water-shed of 200 000 000 galls. to prevent deficiency in a dry year, or a total storage of $200\,000\,000 \times 361 = 72\,200\,000\,000$ galls.

Later computations of the yield of the Croton, in which the gaugings of flow have been continued nearly up to date, show a continued close approximation between the actual average daily yield and the required storage as deduced through the medium of Mr. Stearns' tables based on the Sudbury flow, and the actual storage planned.

The question of trenching the outline of the foundation in order to save the excavation of self-supporting slopes was fully considered, in connection with the general problem presented in the matter of the earth excavation at the dam, and the writer is of the opinion, judging from the experience had with this part of the work, that neither time nor expense could have been saved by such methods. The maintenance and drainage of trenches sufficient for the purpose, which on the limestone side of the foundation would have had to be carried to varying and great depths into the bed-rock, would have proved especially expensive, as well as tedious, and, even with the trenches established and the retaining walls built in them, the resumption of the interior excavation work would have tended to cause delay and confusion, as the excavation and masonry work would have had to be carried on in close proximity in a place where lack of working room was always found to be a hard condition to meet.

The main consideration governing the change resulting in extending the main dam farther into the side hill was the reduction of the height of the embankment at the south end of the dam. The maximum height of the embankment above the level of the restored surface will now be about 50 ft. Another advantage gained is that the hard-pan of the core-wall trench extends to the rock foundation for the full length of the trench, up to the juncture of the wall and the main dam.

* "Suggestions as to the Selection of Sources of Water Supply," by F. P. Stearns, M. Am. Soc. C. E. Report of Massachusetts State Board of Health, 1890.

Mr. Gowen.

The extensive bank of hardpan on the south side of the valley made it practicable to introduce the core-wall and embankment features into the design of the dam, and was, in fact, one great reason for seriously considering this location as feasible and advisable, under certain circumstances, in the beginning.

The core-wall is planned to stop at Elevation 200, as the water in the basin will not rise above this level except for short intervals of time, and the embankment, at this elevation, will be fully 130 ft. thick and well paved.

As to the possible upward pressure due to percolation under the foundations: Mr. Gould alludes to fears on this score as groundless. In this the writer fully agrees with him, but it must not be lost sight of that Mr. Deacon, in building the Vyrnwy Dam, established a series of collecting and discharging drains in the body of the dam, in anticipation of possible percolation tending to influence the structure's stability.

The query in regard to the cement used is pertinent, and a more definite statement is warranted. The Portland cement thus far used is American Portland, Giant Brand, while the term "American cement" alludes to light-burned cement. Of this, large quantities have been used—mostly of the Union and Bridge Brands—the former a Lehigh Valley and the latter a Rosendale cement.

Regarding Mr. Gould's reference to Fig. 2, Plate XLVI, and his criticism of the shape of the rubble stones used for the up-stream facing, it may be said that in the effort to build a face with as small an amount of joint surface (requiring spawling and pointing) as possible, the ordinary rules governing the proportions between the height and width of stones may have been ignored at times. This, it would seem, was warranted when the great thickness and monolithic character of the structure is considered.

The writer regrets that Fig. 1, Plate XLVI, should show a questionable streak near the middle of the unfinished end of the spillway. This streak was due to the wash of surface material from above, and he is glad to assure Mr. Gould that the workmanship at this point compares very favorably with that at any other point of the structure.

Mr. Rafter's account of the use of a rubber packer in connection with the diamond drill is interesting and valuable, and the writer fully agrees with him that through the water pressure tests opportunity is offered for very complete utilization of diamond drill borings.

The results shown in the quotations from the log of the pumping tests are remarkably well defined, and the back pressures obtained and flows traced, at a distance and at comparatively high elevations, are notable, if only as indicating the extent to which such tests can be used in the examination and tracing of particular seams, as well as masses of rock in general.

Mr. Gowen. In regard to Mr. O'Hanly's contention that there was no occasion for going below Elevation — 25 (the surface elevation of the limestone rock) for the dam foundation, and his reference to Professor Kemp's report to justify this position, the writer wishes to say that this report was quoted as being of interest under the circumstances, but that the paper is clear in showing that the investigation of the rock bottom revealed caves, fissures and seams of no inconsiderable size and extent, reaching in some cases to at least 40 ft. below the surface of the rock. In view of this, it is difficult for the writer to understand how at this time a course of action can be criticised which resulted in disclosing these defects which the report indicated as not likely to exist. That much of the foundation limestone was in a state of deterioration was evident, that some of it might have been in process of improvement is perhaps possible, but that the engineer should have accepted the disintegrated, seamy and fissured surface rock as a foundation, in view of a remote future possibility of change for the worse or better, seems indeed a novel idea. An engineer in so doing could certainly not be accused of timidity.

Lack of time prevents the writer from more than following the analysis by which Mr. O'Hanly has determined pressures at various points in the section of the dam, and he is willing to concede that a 1-in. cube of magnesian limestone will stand a crushing strain of many times the proportionate pressure on the dam foundations, but he fails to see any analogy between the crushing strength of a 1-in. cube of compact though not necessarily very hard stone, and the bearing strength of a mass of disintegrated seamy rock, through the mud-filled seams of which some percolation would certainly take place, with perhaps a possibility of washing out the earthy material.

Maximum pressures at the toe were calculated to amount to at least 15 tons per square foot, and the most ordinary prudence would seem to demand a reasonably compact and uniform bearing surface for a foundation.

Another point deduced by Mr. O'Hanly is that the weak point of the section is at Elevation 140, where he concludes that the line of pressure for a full basin falls slightly outside of the middle third. The writer is not disposed to admit this, as his records show that a recalculation of pressures, made some time since, when it was proposed to use the heavy "gabbro" stone for the hearting of the dam, indicates conclusively a well-defined margin of safety at the middle third at that elevation, as well as an excess of sectional area from an elevation above that point and extending to the toe. In this calculation the weight of masonry was taken at 166.67 lbs. per cubic foot. Mr. O'Hanly's assumption of high flow line at Elevation 210 is excessive, and may account for the difference in part of the results.

The triangular section derived from the Rankine formula has its

advocates, but, so far as the writer knows, it has generally been considered for high masonry dams in connection with a solid foundation, and as calculated to stand a possible full upward water pressure upon the base. In the section proposed, with the base at Elevation 40 and the thickness at this point, 240 ft., the area would approximate that of the section adopted for the New Croton Dam, and while there might be no question as to the ability of the ample section provided to withstand the pressure of the water, the efficiency of the proposed cut-off wall and the general effect resulting from the pressure of so heavy a mass of masonry upon a gravel foundation would be questionable. The uncertainty of results and the excessive section used would more than offset the saving of the excavation necessary to carry the other section to rock. Mr. Gowen.

The other triangular section proposed, with a width of base of 204 ft., at Elevation 68, while offering a reduced sectional area, would be open to the previous objections to a still greater degree. At this location it would show as a type of heavy masonry dam 142 ft. high, designed to resist a water pressure of about the same height, and resting on the ground surface.

The question as to whether dams so designed would be safe, raises the somewhat broader question as to whether any high masonry dam can be safely built upon other than a rock foundation. It seems evident that, to-day, practice answers "no."

The remarks of Mr. O'Hanly as to the desirability of adding to the paper details of the cost of the work, and men and plant employed, are fully appreciated by the writer, but such information to be of value must be spread in considerable detail, and would form an extensive paper by itself. The necessity of waiting until the work is finished in order that the information may be complete, and therefore more reliable, must also be apparent.

Vol. XLIII. JUNE, 1900.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

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No. 876.

THE IMPROVEMENT OF A PORTION OF THE JORDAN LEVEL OF THE ERIE CANAL.

By WILLIAM B. LANDRETH, M. Am. Soc. C. E.

PRESENTED FEBRUARY 7TH, 1900.

WITH DISCUSSION.

The Jordan Level of the Erie Canal extends from Lock 50, about 5 miles west of the City of Syracuse, N. Y., to Lock 51, near the Village of Jordan, N. Y., a distance of about 16 miles.

In the improvement of the Erie Canal, under the laws of 1885 and 1886, the work on the Jordan Level was embraced in Contracts Nos. 3, 4 and 5 of the Middle Division. The canal on that level runs for about 5 miles through a summit swamp, where the material is, first, 2 or 3 ft. of muck, then a mixture of marl and clay from 2 to 50 ft. in depth, then cemented gravel. Contract No. 4, extending from "a point 100 ft. west of the Camillas Road Bridge to a point 100 ft. west of the Peru Road Bridge, a distance of 6.31 miles," covers the worst part of the marl and clay formation. This contract was let to John Dunfee and Company, of Syracuse, N. Y., early in December, 1896, and work was begun thereon soon after by Belden and Seely of the same city, sub-contractors, and completed August 1st, 1898.

H. C. Allen, Assoc. M. Am. Soc. C. E., and later, J. H. Grant, M. Am. Soc. C. E., were the engineers for the contractors, and Mr. M. B. Palmer, and, later, the writer, were in charge as Assistant Engineers for the State Engineer's Department.

The Superintendent of Public Works of the State was represented by several inspectors under the orders of J. Nelson Tubbs, M. Am. Soc. C. E., General Inspector of the Department of Public Works.

Believing that a description of the difficulties encountered in the marl beds, and of the methods used in overcoming them, may be of interest to the members of this Society, the following paper concerning the work on Contract No. 4 has been prepared.

The writer's knowledge of the work, prior to November 13th, 1897, is derived from the statements of the various contractors, engineers and inspectors connected with the contract; from the reports of the State Engineer and Surveyor; from testimony taken by the Canal Investigating Committee in the summer of 1898; and from an article* by Mr. George A. Morris, Resident Engineer of the Middle Division.

General Topography.

Nine Mile Creek, the outlet of Otisco Lake, and one of the feeders of the canal, passes under the canal half a mile east of the eastern end of Contract No. 4. White Bottom Brook, the next stream of any size west of Nine Mile Creek, passes under the canal about half a mile east of the western end of the contract; and between those streams the canal runs through a nearly level plain from half a mile to 2 miles wide, bordered on the south by high hills and on the north by rolling country.

The original canal ran along the foot of the hills on the south side of the plain, crossing several small creeks between Nine Mile Creek and White Bottom Brook. These small streams meandered along the plain and emptied finally either into Nine Mile Creek or White Bottom Brook, depending on their position east or west of a low divide.

The present canal was located generally north of the former one and nearer the middle of the plain, necessarily crossing the small crooked creeks several times. As the water level of the present canal was originally below the level of the plain, the natural surface drainage was cut off and the canal prism formed a new channel for it.

*"Earth Slips on the Jordan Level Marl Beds of the Erie Canal," *Engineering News*, December 1st, 1898.

During the season of navigation the water in the canal held the water in the old creek channels above its natural level in places, and extensive swamps were formed on both sides of the canal. When the water was drawn down in the canal, during the closed seasons, the swamps were partly drained; and during a greater part of the year parties excavating marl and clay from the adjacent swamps, for the manufacture of Portland cement, pumped the water from their pits into the prism of the canal.

First Construction of the Present Canal.

From all accounts, the first construction of the present canal was a difficult and costly job. Several attempts were made to drain the marl beds, so that the work on the prism proper could be carried on, and the work was completed finally by building a brush mat on the line of the proposed banks and dumping the material taken from the prism on it. Three separate contractors abandoned the work, and finally the State was obliged to finish it.

The Improvement of 1895, 1896 and 1897.

The plans of the State Engineer and Surveyor for the improvement of the Jordan Level, under the laws of 1895, provided for the lowering of the bed of the canal 1 ft. and raising the banks 1 ft. with the excavated material, thereby obtaining an additional depth of 2 ft. of water. The estimated quantities of material to be moved were those shown by cross-sections of the prism and banks taken during 1895, no allowance being made for the possible re-excavation of material which might slide into the prism.

The estimated cost of the work, by the proposal of John Dunfee and Company, figured on the published list of quantities for Contract No. 4, was $154 471.

Prosecution of the Work.

The water in the level was drawn down as low as possible through waste gates into several streams, but, owing to the accumulated mud in the prism, 2 ft. or more of water remained. To drain the prism thoroughly the contractor cut off all streams flowing into it, dammed off short sections where work was to be started and pumped the seepage water over the banks into the adjacent swamps. One small

FIG. 1.—EXCAVATION OF SOUTH DRAINAGE DITCH, SHOWING MARL FORMATION.

FIG. 2.—PORTION OF SOUTH DITCH, SHOWING BANKS CAVED IN WITHIN A MONTH AFTER THE COMPLETION OF THE DITCH.

stream was carried across the canal in a wooden flume and into an old State ditch leading to a branch of Nine Mile Creek. A great amount of water found its way into the prism of the canal, especially during the months of February, March and April, 1897, and several large centrifugal pumps were operated day and night in order to keep the prism even reasonably dry.

As soon as the excavation of the bottom of the prism was begun, in the soft material, the heavy banks settled, pushing up the mud in the bed of the canal; the banks and slope walls slid into the prism; bridge abutments began to settle, and there was great difficulty in making any progress at all. In several places, cross-sections made one day would show that the bed of the canal was higher than on the preceding day, though in the meantime several hundred men had been excavating mud from that portion. Piles, driven as far as posible into the underlying cemented gravel, would rise several feet in a night and have to be re-driven, though it was found that after being re-driven they did not rise again.

The general plan for holding the slope walls in place, during the early progress of the work, was that of driving short piles, or railroad ties, into the soft material at the foot of the walls, placing toe-beams against them and building a new wall from them to the bottom of the old one; but it was soon found that the pressure of the banks pushed the old and new work into the prism. Sheet piling was also tried, but with the same result. In some cases, where the material in the prism was undisturbed and only a ditch was dug along the foot of the slope wall to receive the row of piles and the toe-beam, the wall would stand, but when the remainder of the prism was excavated the wall and bank slid.

The surface of the cemented gravel was very uneven and its slopes changed abruptly, so that a structure would rest partly on the hard gravel and partly on the soft material, or on the piles driven into it, and would settle unevenly. The berme abutment of the Newport Bridge soon began to slide toward the canal, and, after several ineffectual attempts to hold it in place, it was torn down, a pile and timber foundation was put in and a new abutment built thereon. The piles under the abutment were 57 ft. long, driven as far as possible into the cemented gravel (probably 2 or 3 ft.), but when the backfilling of cinders behind the abutment had been carried about half way to the

top of it, the whole structure began to move toward the canal, followed by the bank and a neighboring hotel.

Timber struts were put across the bottom of the canal from the face of the new abutment to the foot of the slope wall opposite, and the movement of the abutment ceased.

The difficulties encountered during the winter of 1896–97, and the plans proposed for overcoming them, are described in a report* made to the State Engineer by Mr. W. H. H. Gere, Division Engineer on the Middle Division of the canal, as follows:

"In preparing the preliminary estimate for Contracts Nos. 3, 4 and 5, consisting of the Jordan Level, the conditions and difficulties to be encountered to lower the prism 1 ft. were not understood and could not have been anticipated. For about 5 miles the canal is cut through a swamp, the surface of the adjoining land is several feet above the surface of the water in the canal, and upon either bank the material originally excavated was deposited, forming heavy banks. The surface soil is muck, underneath which is quicksand and marl reaching in some places 40 ft. in depth below canal bottom. The surface of the swamp being high enough, was drained into the canal, as no other remedy has been provided; in order to do the work in the prism all these streams had to be closed, thus allowing the swamps to fill with water. As a result the whole mass of marl and quicksand became soapy to such an extent that the material in the bottom of the prism would raise faster than it could be raised with modern dredges. The heavy spoil banks settled, raising the prism and carrying down the towing-path and old slope wall and the bench on which it rested on the berme side. About one-half of the length of the contract has been excavated and the slope-walls built. Just before the opening of the canal, the towing-path settled several feet, and the only remedy available was adopted by bridging with material on hand and planking the surface for a towing-path. The most difficult portion of the work is yet to be done. How to change the conditions to enable the contractors to do the work was a question of serious moment. After a thorough conference with the State Engineer and Superintendent's Department, it was finally agreed that—

"*First.*—The swamps must be drained.

"*Second.*—That in order to form a foundation for slope-wall and to prevent the material raising in the prism at each side from the weight of the heavy spoil banks upon both sides of the canal, it was decided to drive a close row of piles at the toe of the slope-wall through the soapy material into the underlying strata of hard material ranging from 15 to 40 ft., upon which, as a foundation, the slope-wall will be built.

* Annual Report of the State Engineer for 1897, pp. 222, 223, 224.

FIG. 1.—EXCAVATION FOR STRUTS IN MARL BEDS.

FIG. 2.—PLACING STRUTS IN GROUPS OF FIVE.

"In order to drain the swamp, it was decided to construct a culvert under the Nine Mile Creek feeder on Contract No. 3, to give an outlet into Nine Mile Creek, and excavate ditches upon both sides of the canal through the swamp, covered with timber, of sufficient capacity to drain the swamp without allowing it to enter the canal. This work and the piling not contemplated at the time of making the estimate will cost about $80 000.

"The material in the prism in much of the distance completed was quadrupled from raising. Slope and vertical walls and bridge abutments after completion would slide into the canal and have to be relaid, sometimes more than once. The troubles encountered upon this contract are so serious and various, that an attempt at description will give but an incomplete idea of their magnitude, and an approximate estimate of the final cost of the work cannot now be made; but if the result from ditches and pile protection proves all that is anticipated, the work to complete the contract will be comparatively easy and will be done without doubt the coming winter.

"I have prepared the foregoing statement, not as a justification of the cost of the work under this contract, in excess of the estimate made prior to letting the work, but as a matter of history of the most difficult work in all its surroundings upon the canals of this State, and I know that this effort falls far short of doing full justice."

Early in the progress of the work in the marl beds it was found to be impossible to measure or calculate the quantity of material removed from the prism where the bottom was steadily rising, and payment was made to the contractors by the method known on State work as by "force account," whereby they were paid for the wages of the men, and for teams, etc., with 10% added for profit, wear and tear of tools and depreciation of plant.

The culvert under Nine Mile Creek feeder and the drainage ditches were built during the fall of 1897, and no further trouble was encountered from the surface water. As the side ditches were at a higher level than the bottom of the prism, pumping was necessary to free it from ground-water, but to a less extent than formerly.

During the fall of 1897 piles were driven in a close row at the foot of the side slopes of the prism, from boats, over the greater part of the marl beds. Test piles were driven every 100 ft., or closer where necessary, to determine the length of, and line for, the close row, and very good alignment was secured in most cases.

At the close of navigation, in December, 1897, the general condition of the work on the contract was as follows: A part of the prism excava-

tion in the marl beds had been completed; drainage ditches had been dug; long piles had been driven at the foot of the slide slopes over the greater part of the marl formation; a new berme abutment had been built for the Newport Bridge, and the berme abutment for the Memphis Bridge had been underpinned with heavy stone; the greater part of the tow path had been graded; a large amount of old slope wall had been underpinned and topped out up to the new grade, and a new wire fence had been built over several miles of the right-of-way.

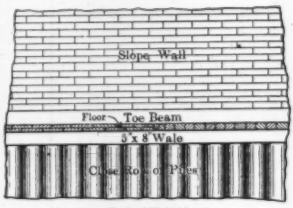
A great amount of opposition was met from farmers owning land through which the new drainage ditches passed, as the State had not bought the right-of-way, and a dozen or more suits had been brought against the sub-contractors for trespass. A test case has since been carried to the Appellate Division of the Supreme Court, and, thus far, the decision has been against the sub-contractors.

Season of 1897 and 1898.

During the summer of 1897 the contractors had delivered large quantities of slope-wall stone and gravel lining by boat, and deposited it along both banks of the canal, for future use. When the prism was drained, at the close of navigation, early in December, 1897, the banks began to slide, especially in that part of the marl beds where they had been loaded with stone and gravel, and a corresponding rise of the material in the bottom of the prism followed.

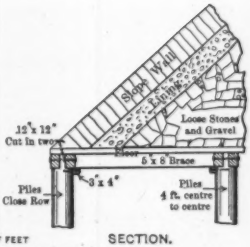
On some sections of the work the long piles driven during the summer remained in place, but where the soft formation was deepest, from Station 360 to Station 410, and at Newport Bridge, they moved several feet toward the middle of the canal after the water was drawn down. The marl and clay in the canal banks would break off with a nearly vertical fracture, settle from 4 to 8 ft., and a mound of mud would rise in the bottom of the canal. For a length of about 2 miles, east of Newport Bridge, the long piles remained in place, and little sliding of the banks occurred. It soon became apparent that the plan of driving long piles at the foot of the walls would not suffice in all cases, and a conference of the officials and engineers of both departments was held, to devise some effective means of construction.

As the result of that conference the following plan was adopted. To drive a close row of piles along the toe of the slope walls, passing



ELEVATION.

Fig.1

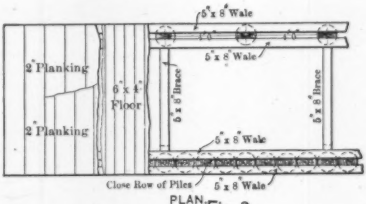


SECTION.

Fig.2

SCALE OF FEET 0 2 4 6 8 10

PLAN OF PILE AND TIMBER FOUNDATION.



PLAN.

Fig.3

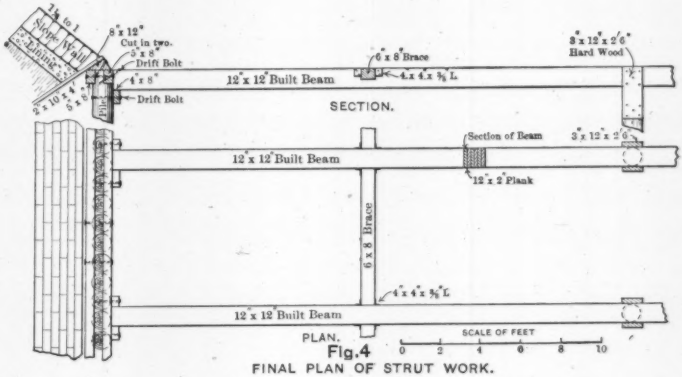


Fig.4

FINAL PLAN OF STRUT WORK.

through the soft material and as far as possible into the underlying hard material; drive a second row of piles in the bank 8 ft. back of the first row, spacing them 4 ft. between centers, excavate between the two rows down to the new bottom of the prism; tenon each row of piles and bolt a 5 x 8 in. waling piece along each side of each row; brace the two rows of piles apart opposite every other pile in the back row, by a 5 x 8-in. timber; lay a floor of two courses of 2 x 12-in. plank on the structure thus formed; bolt a heavy toe-beam along the front edge of the floor to hold the toe of the new slope wall; start the new slope wall and fill behind it with stones and gravel.

In building by this plan, owing to the time necessary to excavate between the piles down to the new canal level, tenon the piles, and put the waling timbers and cross-struts in place, it was found that the back row was forced toward the canal, so as to narrow the width of the foundation; and in some cases the entire foundation moved several feet toward the prism while the slope wall was being built.

It is possible that the plan could have been carried out successfully if more rapid methods of excavating could have been used; but the unstable bank would not support the weight of power excavators, and the nearness of the date for opening the canals for navigation prevented the adoption of any plan for the removal of the banks, which would take time for installation.

After a few hundred feet of foundation had been put in place with only partial success, a conference of the officials of the two departments was held, and the following plan was adopted:

A row of piles was driven on the line of the toe of each slope wall; waling timbers were spiked on each side of each row, and struts of 12 x 12-in. built-beams were placed across the prism at 8-ft. intervals, butting against the front waling timber at each row of piles; a toe-beam was bolted on the top of each row of piles; and a floor for the slope wall was built thereon by driving 2 x 10-in. planks, 4 ft. long, into the material of the old bank. To prevent any vertical movement of a strut, a pile was driven to hard bottom at its middle; cut off 6 ins. below the level of the ends of the strut, and fastened securely to it. Lateral movement of the struts was prevented by placing between adjacent ones braces made of 6 x 8-in. timbers.

The abandoned plan is shown in Figs. 1, 2 and 3, and the final plan in Fig. 4.

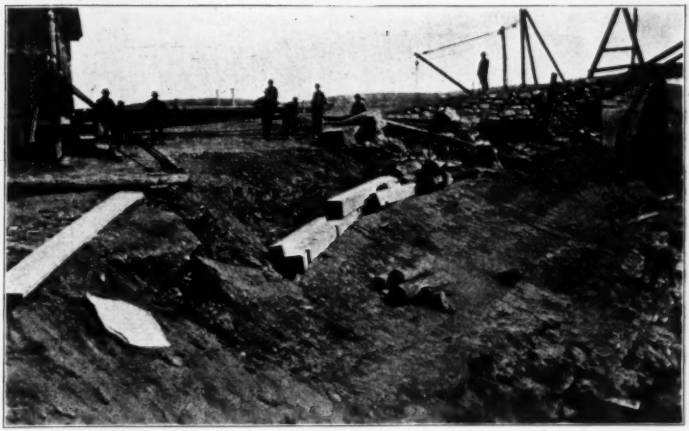


FIG. 1.—SLIDE IN TOW PATH NEAR NEWPORT BRIDGE.

FIG. 2.—SLIDE IN MARL.

The best method of procedure in putting the construction in place, and that followed generally, was to drive the middle row of piles first; excavate trenches across the prism for the struts; put them in place with their middle resting on the driven pile and their ends on temporary mud-sills to prevent their sinking into the mud; drive the side rows of piles; bolt the wales in place; cut the ends of the struts off in line and drift-bolt them to the wales, wedging between the end of the strut and the wale with hard wood when necessary; build the floor and slope wall; and excavate the remainder of the prism as soon as convenient.

To lessen the sliding of the banks during the placing of the struts, five of them were put in place, then a space was left for five more, and a group of five was placed the proper distance ahead; the gaps being filled by a second gang of men.

About 3 700 lin. ft. of canal between Station 360 and Station 410 and 500 lin. ft. near the Newport Bridge were repaired successfully by the final plan. Wherever the piles driven during the former summer had remained near the line of the new work they were used, but in most cases they had moved so far out of line that new ones had to be driven. The long piles were chiefly Michigan pine, from 30 to 50 ft. long, "not less than 9 ins. in diameter at the small end, under the bark, nor less than 12 ins. 3 ft. from the butt." Some piles which had been driven from 2 to 3 ft. into the cemented gravel, when forced out of line by the side pressure, broke off at the surface of the gravel with a sharp, snapping sound. Where the side banks were very soft the piles were driven in a close row, to serve also as sheet piling, to prevent the material in the banks from working under the toe of the new slope walls. In other cases they were spaced 3 or 4 ft. between centers. Loose stones and gravel were used in filling back of the slope walls, the quantity used being determined by a tally kept of the number of loads delivered in wagons of a known capacity.

As before stated, the berme abutment of the Newport Bridge had been rebuilt and braced against the opposite vertical wall during the summer of 1897, and up to the closing of navigation but little sliding of the tow-path abutment had occurred, though the vertical wall in front of it had settled several feet. The tow-path abutment rested on mud-sills at about the new water level of the canal and 10 ft. above that of the bottom of the new vertical wall to be built in front of it.

To keep the abutments in place during the construction of the new vertical wall, and the consequent removal of the temporary struts, a new brace was built between them. The new brace was similar to an oil-well derrick laid on its side, its base covering a space of about 12 ft. x 12 ft. of the berme abutment, and its top pressing against the face of the tow-path abutment high enough to permit of the construction of the new vertical wall. The placing of a new brace, building a pile and timber foundation for the vertical wall, and the construction of the wall itself were completed without any movement of either abutment taking place. Inclined struts were built from the back of the foundation of the new wall to the bottom of the front face of the tow-path abutment to prevent its sliding whenever the main brace was removed. Several small slides occurred in the banks when the water was let into the level, in May, 1898, where no struts had been used, but in all cases, except one, they were repaired before the level was full.

Description of the Materials Excavated.

The marl varies in color from a pure white to a yellowish white. It is handled readily when dry or plastic, but becomes very slippery when wet. It lies in horizontal layers, varying from a few inches to a foot in depth, with a very thin layer of black decayed vegetable matter separating them. The marl deposit, taken as a whole, varies from 2 to 15 ft. in depth, and its depth often changes abruptly.

The clay underlies the marl, and is dark gray in color, with some layers which are nearly the color of common blue clay. It lies in horizontal layers, varying from 1 in. to 2 ft. in depth, separated by a very thin layer of a lighter colored and finer clay, upon which the different layers slide when disturbed. The clay can be handled readily when plastic or dry, but flows like water when saturated. When dried carefully, so as to prevent caking, all of it passes a 60-mesh screen, and about 10% is held by a 100-mesh screen. It shrinks about 10% when dried, and contains from 0.08 to 0.10 of 1% of sharp sand. In some parts of the clay beds there is a layer of sharp sand from 1 to 2 ins. thick between the clay and the underlying cemented gravel.

At the works of the Empire Portland Cement Company, at Warners, N. Y., Portland cement of excellent quality is made by mixing equal parts of marl and clay. The process of manufacture is known as the "semi-humid," the output of the works being 600 bbls. per day.

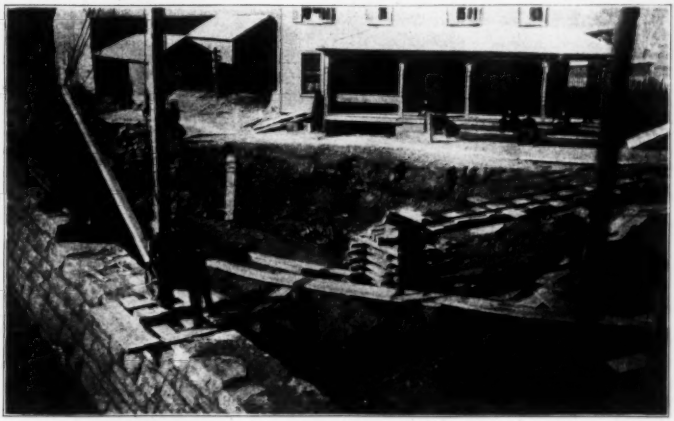


FIG. 1.—SHOWING EXTENT OF SLIDE NEAR NEWPORT BRIDGE, AND CHARACTER OF MASONRY IN NEW VERTICAL WALL.

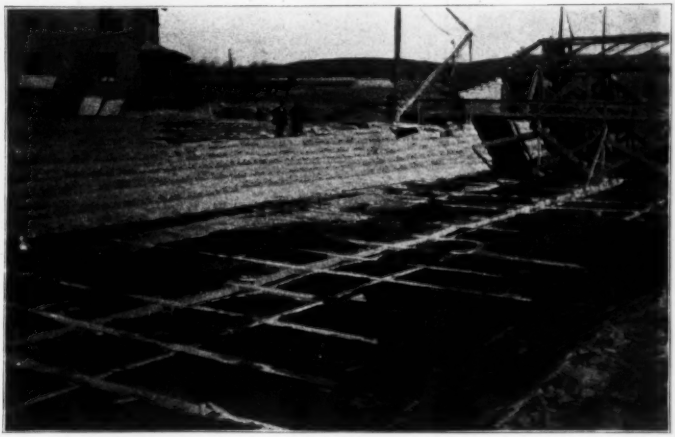


FIG. 2.—STRUTS ACROSS PRISM NEAR NEWPORT BRIDGE, AND VERTICAL WALL READY FOR CONCRETE COPING.

The following analyses of the clay and marl are taken from a description of the works of the Empire Portland Cement Company.*

	Percentage of marl.	Percentage of clay.
SiO_2	0.26	40.48
Al_2O_2 and Fe_2O_3	0.10	20.95
$CaOCO_2$	94.39	25.80
$MgOCO_2$	0.38	0.99
Loss by ignition	4.64	8.50

When saturated, the clay acted like quicksand, and some persons called it by that name; but, to the writer's knowledge, no true quicksand (*i. e.*, a mixture of rounded particles of sand and clay, the sand predominating) was encountered on the work.

In a few places a fine, clean sand was encountered, which was forced up in miniature geysers by the under pressure of the water, and at first sight might be taken for quicksand; but true quicksand, which will flow through crevices and holes where pure sand will not, and which, in the writer's experience, has always been mixed with a finely divided clay, was not found on Contract No. 4.

The cemented gravel consisted of gravel from the size of a pea to pieces 3 ins. in diameter, cemented with a material which, in color and apparent composition, resembled the marl closely. When blasted and dumped in a pile, the cemented gravel re-cemented itself to such an extent that, at the end of a few weeks, blasting had to be resorted to in order to loosen it up.

The writer was informed by the chemist of the Empire Portland Cement Company that the cemented gravel was mostly carbonate of lime, similar to the marl.

The rock was chiefly shale, in ledges from 3 to 10 ins. thick, and was drilled and blasted, except where the cutting was only a few inches thick. Rock, which was very hard when excavated, slaked and turned into mud after a few weeks' exposure to the weather. The cemented gravel also softened up for a few inches in depth when in contact with water for several weeks, making it a very treacherous material for side slopes, unless protected by a slope wall of stone. All the soft material removed from the prism, during the winter of 1897–98, was carried at least 100 ft. back from the canal and spread over the surface of the adjacent swamps to lessen the weight on the banks.

* *The Engineering Record*, July 16th, 1898.

TABLE No. 1.—Estimated Quantities and Prices Bid on Contract No. 4.

Quantities.	Items.	Unit.	Prices.
1....	Grubbing and clearing	Lump sum.	$500.00
1....	Bailing and draining	"	3 000.00
150 000....	Excavation of earth	Cubic yards.	0.275
4 500....	" rock	"	1.00
500....	" masonry	"	0.70
5 000....	Rock channeling	Square feet.	0.10
1 000....	Embankment	Cubic yards.	0.30
17 000....	Lining	"	1.00
1 000....	Puddling	"	0.30
500....	Piles delivered	Lineal feet.	0.10
500....	" driven	"	0.15
30 000....	" at foot of walls, delivered	"	0.09
45 000....	" " " driven	"	0.09
1 000....	White oak timber and plank	1 000 feet, B. M.	50.00
4 000....	Pine " "	"	30.00
246 000....	Hemlock " "	"	16.00
755 000....	Spruce " "	"	17.00
850....	Bridge, culvert and receiver masonry	Cubic yards.	7.00
40....	" " " coping	"	16.00
50....	Vertical wall, Portland cement	"	6.00
1 100....	" laid dry	"	3.50
16 000....	Slope and pavement wall	"	2.47
2 200....	Portland-cement concrete	"	5.00
5....	Asphaltic concrete	"	15.00
10....	Loose stone filing	"	2.00
10....	Portland-cement pointing	Square yards.	0.50
44....	Pavement wall	Cubic yards.	5.00
500....	Cedar posts, set	Lineal feet.	0.40
1 000....	Wrought iron and steel	Pounds.	0.05
10 000....	Cast-iron pipe	"	0.02
3 500....	Spikes and nails	"	0 05
3....	Raising bridges	Each.	300.00
3....	Painting "	"	100.00
	Deduct Materials Furnished by State.		
20....	Dressed coping	Cubic yards.	5.00
170....	Face stone	"	3.50
180....	Backing and vertical wall stone	"	1.50
1 700....	Slope wall stone	"	0.80
10....	Loose stone	"	0.50
2 000....	Hemlock	1 000 feet, B. M.	10.00

The clay was very heavy, weighing from 80 to 83 lbs. per cubic foot; and from 1½ to 2 cu. ft. to a wheelbarrow load was all that a man could push up the plank runways out of the prism. Most of the clay and marl was removed in wheelbarrows, although a clam-shell derrick was used with success on part of the work east of Warner's Bridge, where the material was comparatively dry.

In the marl beds, the slope walls were built of quarry stone, it being found that the cobbles of which the old wall was composed slid on each other, when covered with the marl or clay, to such an extent that they could not be kept in the wall.

TABLE No. 2.—List of Quantities and Unit Prices given in the Monthly Estimate for August, 1898, on Contract No. 4.

Quantities.		Items.	Unit or Lump-sum price.
1	.....	Grubbing and clearing	$500.00
1	.....	Bailing and draining	3 000.00
308 164	cu. yds...	Earth excavation	0.275
25 062	" " ...	Rock "	1.00
526	" " ...	Masonry "	0.70
31 060	" " ...	Earth embankment	0.30
58 915	" " ...	Lining	1.00
276 020	lin. ft....	Piles delivered	0.10
392 650	" ".....	" driven	0.15
84 088	" ".....	" at foot of walls, delivered	0.09
44 688	" ".....	" " " " " driven	0.09
250	ft., B. M..	White oak, timber and plank	50.00
19 650	" " ..	Pine, " " "	30.00
1 055 600	" " ..	Hemlock, " " "	16.00
230 860	" " ..	Spruce, " " "	17.00
705.5	cu. yds...	Bridge abutment masonry	7.00
29.4	" " ...	" " coping	16.00
2 859.9	" " ...	Vertical wall, Portland-cement	6.00
295	" " ...	" " laid dry	3.50
44 800	" " ...	Slope and pavement wall	2.47
339.5	" " ...	Loose stone filling	2.00
4 200	" " ...	Portland-cement concrete	5.00
240	sq. yds...	" " pointing	0.50
44	cu. yds...	Pavement wall	5.00
680	lin. ft.....	Cedar posts, set	0.40
45 240	lbs.....	Wrought iron and steel	0.05
176 440	"	Cast-iron pipe	0.02
38 000	"	Spikes and nails	0.05
3	....	Raising bridges	300.00
3	.....	Painting "	100.00
		Materials Delivered.	
2 000	ft., B. M..	Hemlock timber and plank	12.00
400	lbs.....	Spikes and nails	0.02
		Deduct Materials Furnished by State.	
84.4	cu. yds...	Face stone	3.50
162.4	" " ...	Backing and vertical-wall stone	1.50
9 737.4	" " ...	Slope-wall stone	0.80
64 100	ft., B. M..	Hemlock	10.00
		Extra Work at a Price Agreed Upon.	
108	lin. ft.....	Cement pipe, 22 x 28 ins	1.35
255	" "	" " 18 ins. diameter	0.50
98 412	" "	Piles, 30 ft.—35 ft. long, delivered	0.16
61 522	" "	" 40 ft.—45 ft. " "	0.21
4 047	" "	" 50 ft. " "	0.28
		Extra Work, as Per Resolution of Canal Board Passed September 30th, 1897.	
1	.....	Grubbing and clearing State ditches	1 000.00
1	.....	Bailing and draining " "	500.00
		Extra Work, Paid for with Profit Allowance.	
		Total as Per Estimate of November 1st, 1897	33 337.17
		Extra Work, as Per Resolution of Canal Board, Passed February 2d, 1898.	
1	.....	Bailing and draining White Bottom Brook culvert	1 325.00
1 855	lbs.....	Lead, White Bottom Brook Culvert	0.10
		Extra Work, as Per Resolution of Canal Board, Passed March 31st, 1898.	
1	.....	Raising Newport Bridge	600.00
470	bbls.....	Portland cement used in concrete in excess of contract proportions	2.00
		Extra Work, Per Resolution of Canal Board, Passed February 2d, 1898.	
		General "force account" work	80 982.14

Cost.

The cost of the work in the marl beds, being scattered through many monthly estimates, can only be determined by a laborious examination of the field and estimate books; but the writer believes the following statement thereof to be a close approximation to the actual cost to the State.

Side ditches (on Contract No. 4)	$45 000
Piling to hold banks	100 000
Strut work	116 000
Total	$261 000

As stated previously, the estimated cost of the work on Contract No. 4, at the letting of the contract, was $154 471, calculated on the quantities shown in Table No. 1, and at the prices bid.

Table No. 2 is a copy of the last monthly estimate made by the writer on Contract No. 4, and includes all the work done thereon; but he is informed that a final settlement has not yet been made.

The quantities and prices only are given, except in cases where a lump sum was paid, and on the "force account" work. The official copy of the estimate contains the totals in dollars for each item, but a full copy is not at hand.

The total cost of the work as shown by the foregoing estimate was $606 854.52, and the writer is informed that an estimate by the office force of the Middle Division, from the field notes on Contract No. 4, makes the true amount about $400 less.

A seeming excess of length of piles driven, over the length delivered, as stated in the estimate, is explained by the fact that all piles except those at the foot of the walls were driven at $0.15 per foot; making a real excess of 47 351 ft., in piles delivered. Of this excess of piles delivered, 13 055 ft. was the length of cut-off and the remainder the length of piles on hand at the completion of the work, the property of the State.

General Force Account Work.

The management of the work on "force account" was placed in the hands of a competent General Superintendent of the Department of Public Works who changed the force of workmen as necessary, fixed the rate of wages, and had full control of the laborers.

The time was kept by one timekeeper for the Contractor, one for the Department of Public Works, and one for the Engineering Department; and every day's account was compared and checked that day, material differences in the accounts of the three timekeepers being referred to the Contractor, General Superintendent and Engineer for investigation and settlement.

The General Superintendent and the Engineer worked in harmony and had daily conferences regarding the best methods of prosecuting the work. Payments were made only on the sworn statements of the Engineer and his timekeeper as to the accuracy of the pay rolls; and every precaution was taken against errors creeping in.

The number of men working on "force account" varied from 200 to 600, depending on the amount of work going on.

DISCUSSION.

Mr. Hazen. ALLEN HAZEN, Assoc. M. Am. Soc. C. E.—There are, perhaps, few materials about which more different opinions are held than quicksand. A good definition of this substance is greatly to be desired. This paper contains a definition of quicksand which differs considerably from the idea which the speaker has entertained, and he will present briefly the idea which he has held, with the hope of starting a discussion leading to something more definite upon this subject.

Mr. Landreth's definition of quicksand is: "A mixture of rounded particles of sand and clay, the sand predominating."

The speaker's idea of quicksand is: an even-grained sand, containing for the time more water than would normally be contained in its voids, and, therefore, with its grains held a little distance apart, so that they flow upon each other readily. The sand may be either coarse or fine, generally it is extremely fine. It is the speaker's idea that quicksand in general contains no clay. It may be that some materials contain a little clay, and still act as quicksand; but, if so, that they act as quicksand notwithstanding the clay, and not because of it. A material containing clay particles in considerable quantity is cohesive and impervious. Water may press it out of shape, make cracks in it and rush through it. Under some conditions the whole mass, under heavy pressure, may flow slowly like molasses, but with water it will never make an intimate mixture capable of flowing through small openings and behaving much like water, which is the characteristic property of quicksand.

The sand in a mechanical filter is a good illustration of quicksand. The sand is placed in a tub, with screens or other drainage apparatus at the bottom. The water flows downward through the sand during filtration. Occasionally, the flow is reversed to wash the sand. When the current is downward the sand is firm, and it remains firm after it is drained. If one steps upon it the track hardly shows. When the sand is washed by an upward current, it is lifted by the water, and occupies, perhaps, 10% more volume than it did with the downward current, and in this condition it is suspended in the water, and is so soft that a stick can be pushed into it with but little more resistance than would be offered by so much water.

As the voids in the sand are increased, the friction is greatly reduced, until a point is reached where the friction just balances the excess of weight of the sand over water, and this condition may be maintained indefinitely, the upward current of water just sufficing to hold the sand in a state of suspension.

In this condition it is ideal quicksand. The phenomenon is precisely the same whether the sand is of wind-worn spherical grains or of the

Mr. Hazen.

most angular grains of crushed quartz. In either case the sand is made quick by the passage upward through it of a current of water so rapid that the friction which it encounters more than equals the weight of the sand, and as a result the sand is lifted. If the upward rate were somewhat less, the weight of the sand would exceed the friction, and the sand would not become quick, but would remain solid and firm, as with the downward current.

The upward velocity required to lift a sand in this way is a direct function of the size of the sand grains, and can be computed. The sand used in mechanical filters has an effective size of from 0.40 to 0.60 mm., and the velocity used in washing is such that the friction is more than equal to the excess in weight of the sand over water, but is not three times as great. If it were, some of the sand would be carried away.

Table No. 3 shows the computed velocities at which the friction equals the excess in weight of sands of various grain sizes, or, in other words, the velocities at which the sands will just be lifted.

TABLE No. 3.—Computed Velocities Required to Lift Sands of Various Grain Sizes.

At a temperature of 50° Fahr.

Effective size of sand.	Velocity of solid column of water, in meters per 24 hours.	Velocity of solid column of water, in inches per hour.
0.50 mm.	250	410
0.40 "	160	262
0.30 "	90	148
0.20 "	40	65
0.10 "	10	16
0.05 "	2.5	4
0.03 "	0.9	1.5

This table, perhaps, gives an indication of the reason why quicksands are usually fine sands. The finest mortar sand has an effective size of from 0.20 to 0.30 mm. To lift it, requires an upward velocity of from 5 to 12 ft. per hour, a velocity greater than those which generally occur in the ground water about excavations. That is to say, sand of this coarseness will only act as quicksand where the ground-water currents are unusually strong. With sand 0.10 mm. in diameter, a velocity of only 16 ins. per hour is required to lift it—a velocity which is probably quite common—while the lower velocities of 4 and 1.5 ins. per hour, required to lift sands with effective sizes of 0.05 and 0.03 mm., respectively, are almost sure to exist where excavations are made below the ground-water level in pervious materials; and where sands of these sizes exist they are almost sure to act as quicksands.

Mr. Hazen. There is a condition which may make a sand quick which at first sight would seem to be different from that mentioned, but which in reality is but a variation of it. It is when sand in apparent equilibrium is made quick by a sudden shock or blow. Let us suppose a layer of sand with an effective size of 0.05 mm. and 3 ft. deep, in which the voids are 42%, entirely filled with water. The grains are in a not very stable equilibrium, and this sand is capable of being compacted to 40% of voids. A smart blow or sudden pressure will disturb the equilibrium, and the sand will be suspended by the water which it contains. It will shrink 5% in going from 42 to 40% of voids; and, under the conditions assumed, if perfectly drained at the bottom, half an hour will be required for the excess of water to drain out of it. During this time it will be quicksand. This phenomenon may be seen on many lake shores where the sand is held full of water by capillarity, but in this case the sand is usually coarser, and the length of time that it remains quick is but a small fraction of the above, perhaps only a minute or two, or even less.

A number of samples of material, presumably quicksand, obtained by borings in connection with some of the deep waterways investigations, and which were labeled as mixtures of clay and sand, have been handed to the speaker by Mr. E. P. North. Under the microscope these materials proved to be entirely free from clay, and consist of particles from 0.03 to 0.10 mm. in diameter, having effective sizes of approximately 0.04 mm. Ninety per cent. or more will pass a sieve with 200 meshes per lineal inch. These materials contain a little lime, but, so far as this is the case, the speaker is inclined to think that the lime tends rather to keep them from acting as quicksand than otherwise.

The question may be raised as to the propriety of extending the name of sand to these extremely fine materials. Materials of these sizes occur quite freely in Nature, in which the particles are mostly silica, occasionally with a mixture of hard silicates. Under the microscope they appear precisely like sand. The particles are angular, the arrangement of the particles and the percentage of voids are substantially the same as with coarse sands. The relation of these materials to ordinary sand is much the same as that of sand to gravel, but it is not correct to speak of sand as fine gravel, and it may not be correct to speak of these materials as fine sand. The terms silica dust and sand dust have been suggested, but they imply dryness, and do not seem suited to quicksand. The word silt is also used, but this suggests a somewhat different meaning. Microscopic sand would perhaps be a better term.

Clay is an entirely different substance. Mr. H. W. Wiley,* Chemist of the United States Department of Agriculture, makes the following statement in regard to the properties of clay:

* "The Principles and Practice of Agricultural Analysis," p. 232.

"The percentage of pure clay is about 75% in natural clays, 45% in heavy clay soils, and 15% in ordinary loamy soils. When freshly precipitated by brine it is gelatinous, resembling a mixed precipitate of iron and aluminum oxides. Its volume greatly contracts on drying, clinging tenaciously to the filter, from which it may be freed by moistening. On drying it becomes hard, infriable and often resonant. It usually possesses a dark brown tint, due to iron oxide. Under the action of water it swells up like glue, the more slowly as the percentage of iron is greater. In the dry state it adheres to the tongue with great tenacity. According to Whitney the finest particles of colloidal clay have a diameter of 0.0001 mm. With a magnifying power of 350 diameters, however, Hillgard states that no particles can be discerned." Mr. Hazen.

The clay particles are tens, if not hundreds, of times smaller than the smallest sand grains here considered, and differ from them, both physically and chemically.

It is the speaker's impression that there is a good deal of looseness in distinguishing between clay and microscopic sand. Sand is often so fine as not to be gritty, and when moist it has many of the properties of clay. It differs from clay in its lack of adhesion when dry. A very small percentage of clay, however, makes it adhesive.

The speaker thinks that in many instances microscopic sand, either entirely or nearly free from clay, has been mistaken for clay. So far as he knows, nearly all clay contains more or less microscopic sand, and the percentage of sand may become quite large before it ceases to be called clay. The microscope at once reveals the difference between clay and sand, and there is no good reason for confounding them.

GEORGE W. RAFTER, M. Am. Soc. C. E. (by letter).—In discussing this paper the writer recognizes that Mr. Landreth was not in any degree responsible for the plans adopted, but that he is historian merely of what, for lack of thorough knowledge of the conditions, turned out to be an exceedingly unsatisfactory piece of construction. Mr. Rafter.

As to why this particular construction was so unsatisfactory, the writer will not now attempt to determine. The discussion of that question pertains rather to a broad history of the Erie Canal, in which the results of many years of management of a great public work on political lines are traced to final philosophical conclusions. This part of the subject is of extreme interest and could be expanded indefinitely. Nevertheless, the writer leaves it untouched any further than to remark that the absence of systematic boring records along the Erie Canal probably led to some serious errors of omission.

The methods finally adopted are detailed clearly in the paper. Taken in conjunction with the long struggle against the inevitable, which preceded their adoption, they have seemed to the writer to indicate that, from first to last, this work was conducted on experimental lines purely. Apparently, no one quite grasped the real scope

Mr. Rafter. of the problem presented. In order to indicate the basis for this position, let us outline the physical conditions to be met.

As indicated in the paper, the marl varies in depth from 2 to 15 ft. Beneath this is found soft clay to a depth of 40 to 50 ft. from the surface of the ground. The surface soil is swamp muck.

Such conditions indicate clearly that the margins of excavations should be kept clear of extraneous loads. Nevertheless, as shown by the photographs, this precaution was ignored. Even after a year's experience the contractors were allowed to weigh down the margins of drainage ditches with freshly excavated material. The sliding of the banks of these ditches is therefore merely an illustration that like causes produce like results.

It is clear to the writer, therefore, that the first thing to be done was to clear the margins of excavated material. The next step was to remove the muck above the marl for some distance back on either side of the main channel. After this was done the deepening of the channel, even for several feet, would have been a very simple matter. The slopes would properly have been made flat, in this system of construction.

If embankments over such material are necessary, the proper procedure is to strip the marl for 50 to 100 ft. on each side of the channel, and construct the embankment with a berm 10 to 20 ft. wide on the inside. In this way the writer believes that a canal can generally be constructed through marl without special extra expense, other than for wide right of way. In the present case, if it is deemed necessary to maintain towing paths on the original lines, a timber platform on piles will answer every purpose.

From near the foot of Cayuga Lake to some distance below Mosquito Point, Seneca River flows over marl beds, and from the New York Central and Hudson River Railway Viaduct to Mosquito Point, a new channel was cut in this material about 25 years ago. This channel extends from 6 to 8 ft. into marl, and its banks stand at a slope of about $1\frac{1}{2}$ to 1. In 1858, or thereabout, a new channel for Canandaigua Outlet was also cut through Seneca River marl in the vicinity of Montezuma Aqueduct, which has not given any trouble by the rising of the bottom, such as perplexed the Erie Canal engineers at Warners, in 1896–97. The writer cannot but think, therefore, that a study of the extensive work actually carried out in marl in the vicinity of Jordan Level, would have indicated the proper methods of construction to use in that material.

In regard to the expensive method of piling and cross-bracing finally adopted, the writer understands that it has been only moderately effective. Slides of the slopes still occur. As to the extent of these, it is hoped that Mr. Landreth will give an account in his final discussion.

EDWARD P. NORTH, M. Am. Soc. C. E.—Mr. Rafter's remarks are pertinent, but possibly he has missed, or has not emphasized sufficiently, the principal difficulty, namely, the errors made in planning the work. These errors may possibly be shown to the best advantage by Table No. 4, which gives the Engineer's estimate of quantities and prices, the contractor's bid on those quantities, and the approximate final estimate as given by the author. Mr. North.

It was estimated that more than 6 miles of swamp could be drained and kept free from water at a cost of $1 500. It was this lack of appreciation of the influence of water on marl and clay that nullified the contract; for, after the first season, there was virtually no contract between the State and the contractor, but, to use the words of L. E. Cooley, M. Am. Soc. C. E., "the contractor had a license to prosecute the work at his own price and on his own specifications."

It was well known that with the first canal, built in 1816 to 1825, there was much difficulty on what became known as "the Jordan Level." It was not then as notorious as it was under the enlargement of 1836. The author has stated that three contractors abandoned the work, and that, eventually, the bulk of it was done by the State. The prism of the canal, however, was not thoroughly excavated, and when that was done there was a saving of one-third in the traction necessary to draw a boat through it.

Without an engineer's knowledge of the work to be done and an engineer's plan on which to do that work, it is impossible to cope satisfactorily with difficulties which may arise. This statement is made in emphasis of, rather than in opposition to, anything Mr. Rafter has said. The relations between the specifications and the economical and possibly the successful conduct of the work is rather interesting in view of the recommendation of the Canal Committee of the State of New York, which has recently made a report on the subject. In relation to the crying evil of unbalanced bids, and it has been a crying evil on the canal ever since 1836, the committee proposed to take action by making a schedule of prices, as the French do, in which the price of each item is fixed, and then allowing the contractor to bid either a discount or a premium on those figures. This would apply to all figures; thus, if the price of earth was 30 cents, and of rock 90 cents, a contractor might bid a discount of 1%, or a premium of 2%, and it would affect both the earth and the rock. The plan proposed would apparently eliminate all trouble caused by unbalanced bids, but it would be without influence on imperfect specifications, and the great expense incurred on Section No. 4 of the Middle Division was caused by imperfect specifications as well as lack of engineering knowledge in handling the work. The case was atrocious. The contractors bid $3 000 to drain 6.3 miles of swamp. They were allowed to close the natural watercourses which drained into the canal and thereby turn

Mr. North. TABLE No. 4.—PRELIMINARY ESTIMATE, SUCCESSFUL BID AND APPROXI-

ENGINEER'S ESTIMATE.

Quantities.	Units.	Items.	Prices.	Aggregate.
1	.....	Grubbing and clearing	$500.00	$500.00
1	.....	Bailing and draining	1 500.00	1 500.00
150 000	Cu. yds..	Dry excavation of earth	0.27	40 500.00
4 500	" ..	" " rock	1.00	4 500.00
500	" ..	Excavation of masonry	1.00	500.00
5 000	Sq. ft....	Rock channeling	0.25	1 250.00
1 000	Cu. yds..	Embankment	0.25	250.00
17 000	" ..	Lining	0.60	10 200.00
1 000	" ..	Puddling	0.15	150.00
500	Lin. ft...	Piles delivered	0.15	75.00
500	" ..	" driven	0.10	50.00
50 000	" ..	Piles at foot of walls delivered	0.10	5 000.00
45 000	" ..	" " driven	0.05	2 250.00
1 000	Feet B.M.	White oak timber and plank	40.00	40.00
4 000	"	Pine " "	25.00	100.00
246 000	"	Hemlock " "	16.00	3 936.00
755 000	"	Spruce " "	17.00	12 835.00
850	Cu. yds..	Bridge abutment culvert and receiver masonry	6.50	5 525.00
40	" ..	Coping on above	12.00	480.00
50	" ..	Vertical wall in Portland cement	5.00	250.00
1 100	" ..	" laid dry	3.00	3 300.00
16 000	" ..	Slope and pavement wall	2.25	36 000.00
2 200	" ..	Portland cement concrete	5.00	11 000.00
5	" ..	Asphaltic concrete	10.00	50.00
10	" ..	Loose stone filling	1.25	12.50
10	Sq. yds..	Portland cement pointing	0.30	3.00
44	Cu. yds..	Pavement wall	3.00	132.00
500	Lin. ft...	Cedar posts set	0.10	50.00
1 000	Lbs.....	Wrought iron and steel	0.04	40.00
15 000	"	Cast-iron pipe	0.02	300.00
3 500	"	Spikes and nails	0.05	175.00
3	.....	Raising bridges	40.00	120.00
3	.....	Painting bridges	40.00	120.00
				$141 193.50

* These piles were at various prices.

the swamp into a pond. They knew that the banks were soft and slippery, and yet they were allowed to surcharge them with the material excavated from the canal.

The bottom of the bank on the berm side averaged 40 ft. in width, and the height was about 8 ft.; on the tow-path side the dimensions were somewhat greater. As a result of establishing and maintaining a pond of water behind this bank it was impossible to excavate the material or lay slope walls.

Instead of holding the contractors to their contract they were given an easement of more than $46 000 in the items of digging ditches, and a payment of $2 400 for pumping.

MATE FINAL ESTIMATE ON CONTRACT No. 4, MIDDLE DIVISION, ERIE CANAL. Mr. North.

SUCCESSFUL BID.		APPROXIMATE FINAL ESTIMATE.		
Prices.	Aggregate.	Quantities.	Prices.	Aggregate.
$500.00	$500.00	.....	.....	$1 500.00
3 000.00	3 000.00	.....	.....	4 825.00
0.27½	41 250.00	308 164	$0.27½	84 745.10
1.00	4 500.00	25 062	1.00	25 062.00
0.70	350.00	526	0.70	368.20
0.10	500.00	.....	.....	.....
0.30	300.00	31 060	0.30	9 318.00
1.00	17 000.00	58 915	1.00	58 915.00
0.30	300.00	.....	.....	.....
0.10	50.00	440 001	*	57 400.70
0.15	75.00	392 650	0.45	58 897.50
0.09	4 500.00	84 088	0.09	7 567.92
0.09	4 050.00	44 688	0.09	4 021.92
50.00	50.00	250	50.00	12.50
30.00	120.00	19 650	30.00	589.50
16.00	3 936.00	1 055 600	16.00	16 889.60
17.00	12 835.00	230 860	17.00	3 924.62
7.00	5 950.00	705.5	7.00	4 938.50
16.00	640.00	29.4	16.00	470.40
6.00	300.00	2 859.9	6.00	17 159.40
3.50	3 850.00	295	3.50	1 032.50
2.47	39 520.00	44 800	2.47	101 656.00
5.00	11 000.00	4 200	5.00	21 000.00
15.00	75.00	.....	.....	.....
2.00	20.00	339.5	2.00	679.00
0.50	5.00	240	0.50	120.00
5.00	220.00	44	5.00	220.00
0.40	200.00	680	0.40	272.00
0.05	50.00	45 240	0.05	2 262.00
0.02	300.00	176 440	0.02	3 528.80
0.05	175.00	38 000	0.05	1 900.00
300.00	900.00	4	.....	1 500.00
100.00	300.00	3	100.00	300.00
	$156 821.00			$491 076.16
		Add to this:		
		Extra work.....		33 337.17
		Force account work.....		80 982.14
		Total.....		$605 395.47

An increase of 286¾ over the price bid by the contractor.

See Table No. 2, p. 579.

At first it may seem somewhat brutal to the contractor to say that he should have done his work at the price bid, but the case of the Chicago Main Drainage Canal might be cited, where contracts were taken with somewhat the same mental attitude as that of the contractors on Section No. 4. When the contractors said they wanted relief they were told that no relief would be granted, but that, as their sureties were abundant and satisfactory, if the work was not done by them, it would be done by the Main Drainage Commission and paid for by the sureties. After a time the contractors did the work, and not only without loss, but at a rumored profit of about 50 per cent. In relation to this work, the engineers of the Main Drainage Canal said, with

Mr. North. pride surely, and possibly with justification, that the Chicago contractors, paying $1.50 for their poorest workmen, could have constructed the North Sea and Baltic Canal, for which the German engineers paid 75 cents for their best workmen, and have made more money thereon than the Germans.

The entire science of handling earth and rock (quicksand and hardpan being included with earth), has been advanced more materially by the attitude assumed on the Main Drainage Canal than by any other act by engineers, directors or commissioners during the preceding ten or fifteen years. The speaker thinks that if the State Engineer had said to the contractor, "You are worth more than this can possibly cost, and I will take the last cent you have and finish that work," that the work would have been done at the contract price without much loss.

The literature on quicksand is not, on the whole, voluminous, and greatly lacks defining power. Mr. Hazen's discussion probably gives the most definite information on record, and the most workable theory for quicksands which do not contain clay. But it is immediately seen that a sufficient volume and velocity of uplifting water would turn a boulder bed into a quicksand for the time. The material, however, would become firm immediately on the withdrawal of the upward current. This cannot be asserted of a true quicksand. The relations between the included water and the earth are, in some quicksands, mysterious, and while all quicksands become stable when dry, others, particularly those containing clay, will quake after they are apparently dry.

The late Charles L. McAlpine, M. Am. Soc. C. E., describes a quicksand of the last-mentioned variety very fully in a paper read before the Society in 1881.* In it he says:

"Although its name conveys the idea of a mass of sand, surcharged with water until it becomes 'quick,' or susceptible of easy movement or agitation, suggesting actual life, yet engineers know only too well that this is not the most troublesome member of the family.

"The one that causes the most trouble, and is here treated of, is an argillaceous material containing no silex or grit, comminutes completely, and is usually leaden in color in its natural state, and nearly white when thoroughly deprived of water.

"So free is it from sand that it can be used with good effect in polishing or cleaning silver and the softer metals.

* * * * * * * *

"As far as possible, all traveling over the surface while being thus ditched was prevented, as it agitated the material, and caused it to retain the water more obstinately.

"After a night's quiet rest, and the great withdrawal of water through the ditches, the surface was in good condition for excavating and the material, in the words of the workmen, would then 'shovel like ashes.'

* * * * * * * *

* *Transactions*, Am. Soc. C. E., Vol. x, page 275.

"Care should always be had to withdraw the men and teams at once from any place which indicates that it is again becoming 'quick,' from the disturbing effect of repeated traveling over its surface. Mr. North.

"Nothing is gained by working longer, when this important question of rest in involved.

* * * * * * * *

"A lump of this quicksand, apparently dry, may very often be made 'quick' by a little agitation alone.

"Hard and apparently dry lumps will often become wet and pasty on their way to the dumping ground, so much so as to require additional labor to remove them from the carts."

That all of the above did not preclude, in Mr. McAlpine's mind, such quicksand as specified by Mr. Hazen, is shown by the following quotation:

"It may be assumed generally that the special mobility of such sands depends upon the presence of water filling the interstices of the mass. The mass yields to pressure in conformity to the laws of liquids or semi-fluids, varying with the degree of quickness. The degree of quickness depends upon, *first*, the gravity of the sand; *second*, upon the smoothness of the surface of the particular grains of sand; and *third*, upon the abundance of the water present with it."

Mr. McAlpine may have misnamed the material quoted, but many engineers have met with something very like it, and it is generally called quicksand.

While it cannot be doubted that quicksand may be entirely free from clay, like the samples referred to by Mr. Hazen, it seems equally certain that sand, in the usual acceptance of the term, may be wanting, or that clay which on drying becomes hard and resonant when struck can be washed from some, if not many, samples of quicksand.

JAMES OWEN, M. Am. Soc. C. E.—The speaker has held ideas on the difference between ordinary sand and quicksand, which can probably be illustrated best by a comparison between a pile of loose rock, as the ordinary sand, and a pile of cobblestones, as the quicksand. That is, quicksand is merely rounded sand, water-worn or air-worn, and, while Mr. Landreth has made a distinction by including a proportion of clay, Mr. Hazen seems to have ignored that classification. Mr. Owen.

Some years ago the speaker was asked to report on a foundation for a six-story building, of which a large proportion of the weight was to be concentrated upon one column. Sand, which was thought to be quicksand, had been encountered in the excavation, and it was a question as to whether or not it would be safe to place a large mass of concrete on it. After examining the sand the speaker reported that it was the ordinary wedge-shaped, sharp-edged sand, and that there was no fear of any flow. The building was erected, and no settlement occurred. If the speaker had found that the grains were rounded, he would not have dared to put the structure on it.

To classify quicksand in any other way than by the rounded character of its particles opens the field of speculation, and clouds some-

Mr. Owen. what the broad definition upon which engineers have depended for a number of years. For this reason the speaker believes that it would be well to determine now the difference between quicksand and ordinary sand.

The principle of the wedge-shaped sand is very forcibly illustrated by the practice of the French engineers in building one of the bridges over the Seine. The ends of the centers of the bridge were supported on large boxes of sand. The boxes had loose tops, and the longitudinal frames of the centers rested thereon. There was a faucet at the side of each box, and, when the centers were to be struck, they simply opened each faucet and the sand flowed out slowly with a certain velocity. By this means the centers were lowered very carefully and without shock to the structure.

The sand used was the ordinary cubical sand, and the vertical pressure had no effect on the flow. The sand fell on account of its own gravity alone. If the sand grains had been of rounded form, they would have flowed out with a velocity due to the pressure upon them.

Mr. Hill. GEORGE HILL, M. Am. Soc. C. E.—Some years ago, in excavating for the foundations for the *Mail and Express* Building, in New York City, the speaker encountered an extremely fine micaceous sand, designated by the contractor as quicksand. It was ascertained that the Western Union Building, adjoining, was founded on soil of the same character, and that the foundations were sustaining safely a load of about 3¾ tons per square foot. On the site of the latter building there were driven wells which had been used for water supply some years previously. After pumping for a short time the water began to appear slightly cloudy, and the building began to settle. When the pumping was stopped, the building stopped settling.

The pressure on the foundations of the *Mail and Express* Building is about 3½ tons per square foot, and the building is standing satisfactorily. There was an initial settlement, uniform throughout the entire building, of about § in., compressing the top sand, after which there was no further movement.

Within a year thereafter the speaker designed the foundation for the Pierce Building, with unit loads of 6 tons per square foot, standing on a mixed gravel and sand with rounded edges, about 18 ins. below the water-line, and with no egress for the water. That building stood satisfactorily, without any settlement.

Some years later, the Exchange Court Building on lower Broadway was erected, the excavations being carried about 5 ft. below the water-line. The character of the sand above the water-line was identical with that at the *Mail and Express* Building. As the excavation was carried below the water-line, the sand began to flow in, in spite of rather carefully driven sheet-piling for the foundation pits, the

rapidity of the flow increasing with the depth. In one of the pits, about 6 ft. square, nearly a cubic yard of sand came in during the night. Mr. Hill.

It seems to the speaker that Mr. Hazen's definition of quicksand is more nearly correct than that in common use in New York City, but errs in omitting recognition of the qualification that it is not quicksand unless there is a vent for the water. That is, a material may possess all the elements of quicksand, and yet be perfectly stable and safe to use for foundations until a vent is provided for it. If such a material is called quicksand, the owner imagines that the building must sink into it, is alarmed, and harm is done; if a proper design is adopted for the foundation the material is not, and probably never will be, quicksand. Any sand which is fine is called quicksand by contractors, and they invariably claim that it contains loam. By mixing with water, shaking up, and drawing the water off, the speaker has tried to ascertain whether or not there actually was any loam or clay in much of the material called quicksand, but, so far as he has been able to see, there is no clay, but the sand is very fine. The micaceous sand is excellent for foundation purposes, and, although slightly compressible, is absolutely safe if there is no vent for it. If water is present and there is an opportunity for it to flow off, even though there may not be an excess of moisture, the material will flow.

J. G. Tait, Assoc. M. Am. Soc. C. E.—The speaker is pleased that the discussion has avoided the canal portion of Mr. Landreth's paper, and has brought out the interesting discussion on quicksand. The speaker wishes to correct any false impression which Mr. North's remarks might convey to any member not familiar with the contracts and specifications of the $9 000 000 canal improvement. Mr. North states that the contractor got $46 000 extra for bailing and draining, which he should have been made to do for the original bid of $3 000. The $46 000 was paid to the contractor at the very low excavation price of 27½ cents per cubic yard for material removed in a swamp when constructing side ditches at a loss, and these should have been estimated originally by the engineer, therefore this sum was not a present. Mr. Tait.

There was a great deal of extra work on this contract, which should, and usually does, represent some profit, but the sub-contractors, who got these supposed benefits, are bankrupt to-day. The unusually good quality of the work done, the favors demanded of the contractor, and the final non-payment for work done, through selfish, incompetent State officials, made the Erie Canal experience very costly for the majority of the contractors.

The competition in contracting to-day causes very low prices, and in a lengthy or obtuse specification, a contractor, to get the work, has to take the cheapest interpretation of what he is to be required to

Mr. Talt. do, and bid accordingly. Nearly all the contractors with whom the speaker is acquainted are men who will accept the loss due to a misinterpretation, and who will do the work required by the specifications and contract, but when, in a contract calling for thirty-nine items, one alone of which, in a total of $154 000, increases from $3 000 to $49 000, and is decidedly the fault of the contract and plans, the speaker cannot understand how an honest or fair-minded man, particularly one familiar with all the conditions, can state that the contractor should have been made to do this enormous necessary amount of extra work for nothing.

Instead of taking from or injuring a contractor who has so much with which to contend, the engineer and contractor should both work together for the good of the work, the engineer seeing that he gets his money's worth, but at the same time not using his power to get something for nothing, an old-time policy not followed by all engineers.

Quicksand, with which the speaker has had considerable experience during the past twelve years, is such a bugbear that when contractors encounter any kind of moist sand it is quite natural for them to call it quicksand, as Mr. Hill remarks.

Any sand, even the sharp angular kind, will run, even if is under a head of only one foot, but if it contains no clay, or if the grains are not rounded, it can be controlled readily by sheeting and pumping. On the other hand, if the sand is fine and rounded and contains clay, great trouble is experienced. Even in cases where the sheeting is driven far below the bottom of the excavation, this material may rise in the center and cause the sides of the excavation to cave in, thus producing excessive or unbalanced pressures on the sheeting.

Mr. Whinery. SAMUEL WHINERY, M. Am. Soc. C. E.—It would be very desirable, if it were possible, to have a correct and comprehensive definition of quicksand, but the speaker's experience has been (and he thinks it is the experience of many others), that, after settling upon what seemed at the time to be the proper definition, the very next case is likely to contradict it entirely.

During the construction of a railroad in Western Indiana, some 30 years ago, in making a cut not far from a stream, but well above its water level, quicksand was encountered by the plows and scrapers and stopped the work at that point. In investigating in a rude way it was found that 10-ft. fence rails could be pushed down for their full length quite readily without reaching the bottom of the quicksand.

The sand was very fine, and was almost free from any admixture of clay or other foreign matter. Apparently, there was no possibility of the existence of the conditions to which Mr. Hazen refers, that is, of its being buoyed up by a rising current of water. It is true, the

sand was filled with water, but this water was, apparently, in a quiescent state. There were no springs known to exist in the vicinity. The depth of the deposit was not ascertained, but it seemed to be contained in a pocket surrounded by impervious clay. The case was dealt with by beginning the excavation at the lower end of the cut, where the grade was lower, and working up the grade with deep side ditches until the deposit was reached. The water in the sand then drained out, the material became quite hard, and was then excavated with plows and scrapers. The sides of the cut were dressed to the usual slope, and they have stood quite satisfactorily ever since. Mr. Whinery.

L. J. LE CONTE, M. Am. Soc. C. E. (by letter).—This paper is full of information not often found in print. Engineers are generally chary about publishing accounts of their troubles in practice, and rightly so, because the irresponsible critic is the most active of all. Mr. Le Conte.

When working in treacherous ground, such as described in the paper, the engineer is often at his wits' end to know the best way to meet existing conditions. In many instances it seems as if Nature took a malicious delight in defying all the requirements of the best laid plans and specifications, based on the most trustworthy information.

The troubles depicted so graphically by the author excite the interest and sympathy of any engineer who has had the misfortune to be caught in such a trying position. The simple and effective methods adopted to overcome local obstacles are certainly highly commendable.

The laws of hydraulics and hydrostatics have been well developed by experimental investigators, but the laws governing the dynamics and statics of mud have yet to be formulated in practical shape. In June, 1826, at "Chat-Moss" on the Liverpool and Manchester Railroad, 4 miles of embankment cost nearly $150 000, and took 7 months to complete. The indomitable pluck and tireless energy of the engineer, George Stevenson, prevailed finally and left a monument to his name.

The writer was interested in a case which was particularly trying, and, at first sight, seemed to be impossible of solution.

A proprietor desired to improve a tract of marginal marshland by raising it well above the effects of tide water in a lake. The marshland was underlaid by plastic blue mud which rested upon a solid bed of yellow clay hardpan, the latter having a natural slope of about 13% toward the lake. The mud, at high-water line, was about 40 ft. deep, making the outlook rather unfavorable. The case was complicated still further by the proximity of a deep-water channel and a system of tidal sluice gates in front of, and parallel to, the shore line. This channel could not be encroached upon by pushing out the marsh mud under the pressure of the proposed new filling. The problem was, nevertheless, solved successfully as follows:

Mr. Le Conte.

First.—A trench 4 ft. x 12 ft. wide was excavated along the irregular line of high water or outer edge of the marshland. The material taken out was deposited on the edge of the trench next to the high land, the top width of the bank being sufficient for two runways for wheelbarrows.

Second.—The work of filling in with heavy sandy material was then begun, at and along the segregation line, advancing gradually toward the lake. As the filling progressed, of course, settlement commenced, and the material in the bottom of the trench began to rise up. A force of 100 laborers and excavators had all they could do to excavate and remove the material from the rising bottom as fast as it came up. This process was continued until the entire marshland was filled in solid down to hardpan, and all settlement had stopped. It is a notable fact that the total yardage thus taken out from the bottom of the trench was approximately the same as the total yardage of heavy material in the filling below the level of the marshland, namely, some 140 000 cu. yds., and yet, after the completion of operations, the dimensions of the trench were the same as in the beginning.

It would appear, therefore, that the trench simply afforded a natural vent for the escape of imprisoned mud compressed by the weight of the advancing filling deposited on the marshland. By this means these lands were filled in successfully without crowding the mud into the channel-way in front of the property to any appreciable extent.

Mr. Richardson.

CLIFFORD RICHARDSON, Assoc. Am. Soc. C. E. (by letter).—It may be of interest, in connection with the discussion of the subject of quicksands, to present some data in regard to the actual size of the particles in several such sands which have recently been examined in the New York Testing Laboratory by the writer.

These sands were from the following sources:

No.	Source.
	NATIONAL CONTRACTING CO., BOSTON, MASS.
11 541	Neponset Valley Sewer, Section 26, Station 3 + 50. "When wet, extremely difficult to handle."
11 542	Neponset Valley Sewer, Section 26, Station 2 + 80. "Found under peat. Very troublesome to excavate, and difficult to hold trench in line and grade."
11 544	Neponset Valley Sewer, Section 26, Station 1 + 37. "Very difficult to handle. Squeezes trench badly."
	H. P. EDDY, SUPERINTENDENT OF SEWERS, WORCESTER, MASS.
30 723	Greene Street Sewer }
30 724	" " } see *The Engineering Record*, March 24th, 1900.
30 725	" " }

For comparison with these sands, the finest ground limestone produced in a Danish tube mill, all of which passed a 200-mesh sieve, was examined.

Mr. Richardson.

The sands, Nos. 11 541, 11 542 and 30 725, under the microscope, were seen to be very clean, and to be made up of extremely sharp grains with no clay. Sands Nos. 11 544, 30 723 and 30 724 were equally sharp, but carried a small amount of clay, amounting to less than 1%, and not subsiding from water in a week.

The voids in the hot sand, compacted thoroughly in a 100-c. c. cylinder by shaking and tamping, were determined, where sufficient material was available, and from them the volume-weight per cubic foot. These sands were finally sifted on sieves, and elutriated by the beaker method. In this way they were divided into grades of particles of different sizes. The results are shown in Table No. 5.

Sand No. 11 541 has a wide variation in the size of its particles; consequently, it has low voids and high volume-weight. Sand No. 11 542 would probably show like characteristics, but, with a somewhat wider grading, somewhat lower voids. Each of these sands contains a very considerable amount of grains of high hydraulic value, as do the Worcester sands, Nos. 30 723 and 30 724, with voids much like those of sands used in ordinary mortar—36.7 and 34.7 per cent.

The most interesting sands are the two extremely fine ones, Nos. 11 544 and 30 725. They are of very uniform grading, the majority of the particles being of sizes within very narrow limits. The resulting voids are, in consequence, what are usually found under such circumstances, about 40 per cent.

TABLE No. 5.

Nos.		11 541	11 542	11 544	30 723	30 724	30 725	Finest ground limestone.
Voids in hot compacted sand		29.3%	.....	40.2%	36.7%	34.7%	.....	39.3%
Weight per cubic foot, in pounds		117.2	.....	99.1	103.8	106.1	.....	100.4
Sieve.	Diameter, in millimeters.							
.....	0.035	19.2%	11.2%	65.5%	47.2%	11.6%	79.7%	63.5%
.....	0.065	7.9%	14.2%	13.7%	19.6%	11.4%	9.5%	17.7%
200	0.09	18.9%	19.6%	16.8%	11.2%	9.0%	9.8%	18.8%
100	0.17	34.0%	22.0%	3.0%	13.0%	39.0%	1.0%	.....
80	0.23	11.0%	8.0%	1.0%	4.0%	12.0%	.....	.....
50	0.31	7.0%	16.0%	.....	3.0%	10.0%	.....	.....
40	0.50	1.0%	4.0%	.....	1.0%	3.0%	.....	.....
30	0.67	1.0%	2.0%	.....	1.0%	3.0%	.....	.....
20	1.00	.....	2.0%	.....	.....	1.0%	.....	.....
10	2.00	.....	1.0%	.....	.....	.....	.....	.....
Greater than	2.00	.....	2.0%	.....	.....	.....	.....	.....

The sizes of the grains of these sands are smaller than those in the ground limestone dust, that is to say, than the finest Portland cement.

Mr. Hazen's claim that quicksand depends more for its peculiarities upon the size of the particles and upon their small hydraulic value

Mr. Richardson. than upon any other characteristics, round shape of grain, presence of clay, etc., seems to be confirmed.

It would be of interest to examine the quicksands encountered in the Erie Canal in the same way, and the writer would be glad to do so if samples from that locality, and from any others where such sands are met, can be furnished.

Mr. Cooley. L. E. COOLEY, M. Am. Soc. C. E. (by letter).—Mr. North has referred to the work on the Chicago Main Channel. The treacherous ground on this work extended over 6 to 8 miles, and included a wide variation in the material, of which the amount was several million yards. The history of this work has shown that by hydraulic dredging, or by drainage, large channels can be cut through such material as cheaply as through stable ground. To present the subject, however, opened so wide a field and involved so much labor, that the writer has been disposed to defer any remarks until some more zealous member had presented the Chicago experience.

Quicksand has come to be a broad generic term referring to any material containing usually a large proportion of silicious grains, as distinguished from mud, or other material with a large proportion of clay which shrinks and cracks on exposure. All these materials are stable when the excess of water is removed. Some of them are so finely divided or contain so much clay that they drain or leach out very slowly. Others are so coarse or drain so freely that they can hardly be called "quick," but have been called "alive" under conditions, usually temporary or special. Again, some materials, when once handled, are comparatively stable under new conditions of saturation. That material should be "quick," is due to a condition with respect to water, rather than any quality inherent in the material itself. A monograph is required to discuss fully the subject—"quicksand."

A large range of sands is mobile under the conditions cited by Mr. Hazen; in fact, very coarse material would be unstable under upward flow. The writer has sunk hundreds of piles "butt down" by means of a water jet and without guides or hammer. The material around the pile was kept "alive" by "weaving" or partial rotation. Fishermen about the lakes set poles in 20 ft. of water, or deeper, by "weaving." The material is not necessarily "quick."

Running sands are largely due to the escape of contained waters; to crevasse action rather than any flow *en masse*. Some initial restraint will often prevent this until partial drainage has insured against displacement. The marginal deposits of the Missouri, below mid-stage, are usually quite fine, and become "alive" under the driving of a survey stake, or very active tramping, and the writer has seen the same on the beach of Lake Michigan. Such sands will drain very quickly, almost instantly, and have then no suggestion of instability.

Passing back from the immediate shore of the Missouri to places

sheltered from the high-water current there will be found deposits of finer sand mixed with "gumboe," and these remain "quick" for weeks or months, according to the proportion of clayey material; and finally, this may be so much in excess as to make virtually a mud deposit which cracks on drying. Most of these deposits do not become unstable under subsequent high-waters. Even the yellow bluff deposits, which are very porous (roots penetrating deeply), and which remain perfectly firm under ordinary rain, will become saturated and temporarily "quick" under heavy downpours. This material contains enough clay to puddle, and can be burned into brick. Mr. Cooley.

The study of the deposits of a stream like the Missouri and the source thereof in its water-shed, from the mobile, alkaline clays of the "bad lands" to the coarse granitic sands and the lavas, gives a glimpse into the genesis of the whole tribe of quicksands.

The sortings of the waters in the working over of glacial grindings differ only in age and source of material. Localities may add the results of animal and vegetable life in the marls and mucks. That these deposits should be, or should continue to be, unstable, is largely due to lack of drainage.

On the Chicago Canal were some 2½ miles of muck and mud deposits from 10 to 30 ft. deep, which were removed by hydraulic dredging to a width considerably greater than the channel. The underlying material was dredged and placed on the berm to form the proper banks. The prism was then pumped out and completed "in the dry" in perfect security. In carrying a levee line through this material it heaved in places for 100 yds. distant. Very little seepage occurred, though the Desplaines River was scarcely 100 yds. away, with nearly 40 ft. of head over grade at high water.

For about 1½ miles there were also deposits of muck, marl and mud, in which an iron rod could be shoved down. In the original construction of the Illinois and Michigan Canal, the towpath had been carried on a trestle, and in the deepening of the same the area was a quaking bog, and the prism of the canal called for much redredging in its formation. Various expedients were resorted to at the outset. A part was handled by hydraulic dredging. Eventually, it was discovered that the material could be slowly relieved of its excess of water by continued pumping, and that it could then be handled with the greatest facility by ordinary methods.

Another 1½ miles contained beds of indurated material and heavy beds of quicksand. The most vexatious of this acquired the local name of "bull-liver." No treatment could be decided upon. Meantime, continued pumping had so relieved the situation that, as the work progressed, the "quicksand" came to be the only easy part of the excavation. The writer is of the opinion that there were a couple of miles more of "quicksand" that never showed itself to the con-

Mr. Cooley.

tractor. The writer has waded through the gray, fine and dry dust well down in the channel on a hot day, when it seemed to be a section of the arid regions. Systematic drainage had changed its character before it was reached.

Water was the great bugbear of the Chicago Main Channel, and the writer was substantially alone in advocating its construction "in the dry." Although great difficulty was anticipated from this source, and elaborate consideration given to the handling of water, it proved to be entirely overestimated. This great canal, cutting all the strata for 28 miles to a depth of 35 to 40 ft., hardly produced enough water to supply a city of 100 000 inhabitants, after the ground-water reservoir had once been drained; and all the run-off from 700 miles was carried within less than ¼ mile for 20 miles on one side and the Illinois and Michigan Canal within a like distance on the other for the entire length. There were 9 miles of rock, carrying some water, at localities, and 7 miles of clay; but there were 12 miles with all the variety of formation that can be imagined in an ancient stream bed of the glaciated regions.

The use of gravel, hay, sods, brush and similar material in minor excavations to check running sands until they can drain, is well understood.

Wells may be very effective in draining out ground-water, and it is surprising how little additional water may be developed by an extensive cutting over that encountered in an individual well. Wells are sunk in treacherous ground on a curb, and the sinking of brick wells in India is a very ancient practice. The semi-nomads of the arid regions of Nubia have, from time immemorial, sunk wells for their stock, in depressions and draws, through wind-blown material and fine wash. A hole is scooped out and lined with a thick rope of twisted twigs and brushwood which is added in a continuous coil at the bottom as the excavation descends. Such wells reach depths of 25 to 30 ft. and more. Another device of the Nubians is a flexible or woven bucket which can be carried on the pommel of a saddle; and cooking is even done in baskets by means of heated stones.

There appears to be no reason why it should not be feasible to remove ground-water from extensive areas by systematic pumping from a series of driven wells. After the reservoir is once emptied, it will usually be found that the constant flow is limited and much less than ordinarily supposed.

Seepage from adjacent ditches and streams, even when new strata are cut, is usually much less than supposed. With roily water, stream beds choke or puddle quickly, and percolation becomes very slow or ceases entirely. Unless strata are very coarse and open, the rapidity of flow and the amount are generally overestimated if the distance be considerable, and the pumping item in extensive cuttings need not usually be a large element of cost.

A word as to specifications and unbalanced bids to which Mr. North has alluded. Mr. Cooley.

The writer's views are on record in regard to the character of the Erie Canal specifications in the report of Governor Black's Committee of Investigation of expenditures under the "Nine Million Act." There is nothing in these specifications to be commended, and the French device to prevent unbalanced bids is an attempt to mitigate the evils of a system which in itself is fundamentally wrong.

The Engineering Committee of the Sanitary District of Chicago, of which the writer was Chairman, spent weeks with the aid of its Engineering Staff and its Law Department, in the consideration of specifications and contracts. The endeavor was to so frame these instruments as to conform to equity decisions by the Courts; to make the duties of the engineer purely ministerial; to reduce the bid items to the lowest number practicable and generally to leave as little as possible to future interpretation. The idea was to frame a contract for specific performance, rather than a franchise for doing some unknown thing on a "piece-work and material" schedule of prices, in which the size of the pieces and the amount of material were to be determined later by discovery and by the varying idiosyncrasies of officials and the astuteness of contractors. The essence of such a contract is a knowledge of the conditions, as it is, indeed, of all contracts in the last resort of equity. The obtaining of this knowledge is a presumption of official responsibility, a requirement for public protection; and a neglect of such duty cannot be construed to work a hardship to the contractor.

This does not mean that the contractor is to base his bid on the official data in regard to quality of material, but it does mean that the work is to be so far exhibited that the contractor can form for himself a proper judgment of cost within the limits of a reasonable risk. Pumping, bailing, draining, clearing, grubbing, mucking, plant, material, supplies and a thousand things are all incident to a main purpose which can be sufficiently exhibited in a very few items, capable of reasonably close measurement for purposes of current compensation and final estimate. The specification should be full, and there may be qualifying clauses against abuse in current estimates, but which do not affect the final.

With few items, and these reasonably well ascertained, a little intelligence will prevent unbalanced bids and administer contracts to a successful conclusion, provided they are actual contracts and not permits on a "piece-work and bill of material" basis.

WILLIAM B. LANDRETH, M. Am. Soc. C. E. (by letter).—The problem presented in the improvement of the canal through the marl beds was that of deepening an existing channel and not of building a new one. Owing to the high and wide spoil banks formed by the material Mr. Landreth.

Mr. Landreth. excavated in the first construction of the canal, the general plan proposed by Mr. Rafter, a wide channel with flat slopes, if adopted in the 1896 improvement, would have proven more expensive than the timber and pile construction.

No slides have occurred in any portion of the work described, where struts were placed across the prism. Several careful examinations of the timber construction, made by the writer during the past winter and spring, show that no movement of the struts or slope wall has taken place.

When the water was let into the canal in May, 1898, a portion of the tow-path bank, where only short piles had been used under the slope wall, slid into the prism. This slide occurred in front of the Empire Portland Cement Works, and was probably caused by the vibration of heavy engines and grinding mills in the works adjacent to the tow path. Bids were asked for in November, 1899, for the repairing of this slide, upon plans identical with the pile and timber construction used in 1897–1898, but on more stringent specifications regarding the re-excavation of material.

After several competent contracting firms had examined the locality, plans and specifications, only one contractor submitted a bid. The contract was awarded to him, and the work has been completed in a satisfactory manner for 12% less than the engineer's preliminary estimate.

The price bid for bailing and draining on the last contract amounted to $12.50 per lineal foot of prism, and, taking the length of prism in the marl beds on the former contract as 2 000 ft., the bailing and draining on that contract would have cost $250 000, at the same rate per foot.

The 1899 work, done by contract, on the same plan as the 1897 work and under the direction of the same engineer, cost about 20% more per lineal foot than the 1897 work done under "force account."

The case of the Canandaigua Outlet, cited by Mr. Rafter, is not a parallel case with the marl bed work on the Jordan Level; the former being a dredged channel from which the water is never removed.

The drainage ditches, noted in Mr. North's discussion are of permanent value to the State by lessening the saturation of the canal banks and preventing the surface water from entering the prism.

A calculation of the quantities on Contract No. 4, based on the system of unit prices mentioned by Mr. North, shows that there would have been a difference of about 1% in favor of the contractor, provided the prices estimated and bid had been the same as by the old method.

The discussions on quicksand treat the subject from scientific and practical points and have added materially to the literature thereon, thus attaining one of the objects in view in the preparation of the paper.

Vol. XLIII. JUNE, 1900.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

No. 877.

ADDRESS AT THE ANNUAL CONVENTION, LONDON, ENGLAND, JULY 2D, 1900.

By JOHN FINDLEY WALLACE, President, Am. Soc. C. E.

The profession of Civil Engineering, as we now know it, is a child of the nineteenth century. The conception of that child, however, occurred with the earliest dawn of human history. Among the earlier engineers we find mention of Tubal Cain, who followed the mechanical branch of the profession; Noah, who devoted his attention to ship building; Joseph, who achieved great success in the construction of grain warehouses and irrigation works; the unknown Egyptian who conceived, designed and constructed the pyramids, and in whose honor, for all we know, the celebrated figure of the Sphynx may have been erected; Moses, who devoted his energies and lifetime to the transportation of a nation from one section of the globe to another, also giving some attention to hydraulics; and Joshua, who followed a similar line of work and also utilized his engineering talents in the overthrow of Jericho by means of the theory of rhythmic oscillation.

The ancient engineering profession, by whatever name we may call it, probably achieved its highest distinction during the existence of the Roman Empire, through the efforts of the legion of engineers

whose names have sunk into oblivion but the remnants of whose works remain until to-day. The profession, however, has only reached its present strength and vigor during recent years. It would be useless to attempt to name, in the short time at my disposal, the typical engineers of the closing century.

It has been said that the measure of civilization is the measure of success attained in the preservation of food products. It seems that the truer exponent of modern civilization is transportation; for, while the Hebrew wanderer in Egypt was able to preserve his food products for a period of seven years, and his co-workers preserved in their mummies grains of wheat, which, thousands of years afterward, were transported to the fertile fields of Oregon and have since germinated and yielded the famous Mummy Brand of wheat, for which that region is celebrated, neighboring tribes and nations starved, because the only means of transportation in those days consisted of caravans of camels by land, and the shallow vessels which navigated the Mediterranean without a compass.

No one agency has been a more potent factor in the advancement of civilization than improved transportation facilities, and the profession of civil engineering has been the force which has conceived, designed and executed the works and machinery which have furnished the world with these facilities. There is hardly a branch of the profession which does not touch on the principle of transportation. The construction of steamships, harbor and river improvements, docks and wharves, canals, water supply, sanitation, manufactories, the utilization and transmission of power—in fact, it is difficult to name a branch of the profession which does not, either directly or indirectly, bear on the subject of transportation.

The great engineers of this century have been responsible and should receive credit for the conception, design and execution of these works, without which the Americas, Asia, Africa, Australia and the isles of the sea would still be among the uncivilized, unknown or inaccessible portions of the world. While it is true that modern civilization, as applied to those portions of the globe to-day, is only partial and imperfect, the veil of obscurity has been lifted, the door has been opened, and modern civilization, guided by the hand of the civil engineer, has only to step forward and take possession of the heritage of mankind.

It behooves us, however, as members of this most noble profession, to realize the responsibilities resting upon us and appreciate the magnitude of the labors before us, and to prepare ourselves for the task; and in doing this to carefully observe our past failures and shortcomings. We should realize that our future success depends upon our full knowledge and appreciation of our past errors. We should discover those qualities in which we are lacking, and endeavor to supply and supplement our deficiencies, in order that we may be better fitted to assist our profession in successfully following out its manifest destiny.

The pioneer engineers of the century were strong, rugged characters, educated in the school of adversity, who lived close to Nature's heart and were keen observers of natural phenomena. They possessed strong imaginations, daring originality, faith in their convictions. These pioneers were followed by a class of engineers who were the students of their predecessors, and who supplemented their own talents and advantages by a study of the methods and the fund of knowledge accumulated by those who lived before them. This latter class has been followed by a generation of engineers who have had the benefit of the larger fund of accumulated engineering wisdom and experience, supplemented by academic and technical education; until now, the young man choosing the engineering profession has before him opportunities, easily accessible, of acquiring the highest scientific and technical education, and entire libraries devoted to the description and elucidation of engineering works and methods, from which to gain an accurate knowledge of the great works which have been accomplished by other members of the profession in all countries, and from which he can derive material for the formation of his own plans. From this vast storehouse of accumulated facts, easily accessible throughout the world, he should be able to steer clear of the errors and avoid the failures of older engineers, and should be able to achieve a higher measure of success.

It has generally been understood that mathematical ability of a high order is a fundamental requisite to success in the engineering profession. This is not necessarily true, as it frequently happens that the person in whom mathematical ability of the highest order exists, is deficient in other essential qualities. While the successful engineer should be a good mathematician, should be a man of integrity, industry

and resources, there are other qualities which cannot be ignored. The engineer who constructs a bridge, no matter how carefully he may design and construct his masonry foundations and steel superstructure, would not be considered an engineer, in the true sense of the term, should he neglect the hidden vein of quicksand, which might eventually cause the destruction of the entire bridge.

Among the underlying qualities necessary to the highest success in our profession are: the ability to accurately observe natural phenomena; a faculty of mentally picturing things in their proper relation to each other; good business judgment and sense; a careful study of business methods; tact, which to some men means so much and to others so little; personal loyalty, not only to the enterprise in which he is engaged, but also to his immediate superior; and, above all, the engineer should not neglect the fact that "The proper study of mankind is man."

In the technical schools of the present day much time is spent in considering the strength of materials, the applied mathematics in the investigation of forces; but nothing in relation to the mental and moral constitution of man, which is the live material that engineers must use in fitting Nature's raw products to the work in hand. There is too much of a tendency among engineers to-day, as ever, to consider the profession entirely a materialistic one, and to confine the education of the young engineer to a knowledge of materials and their uses, to the exclusion of the mental and spiritual.

The prevailing law of matter is inertia. The prevailing principle of the engineering profession is life. The application and direction of the life principle to matter, in order to overcome its inertia, change its form and adapt it to our needs, is the problem. Neither factor can be ignored, above all the life principle. There is too much of a disposition on the part of our profession to value too highly what are the tools of our trade; to forget for a moment that they are simply tools, and fail to appreciate the energizing factor and the mental make-up of the engineer.

It is not the most expert bookkeeper who becomes the most successful banker or merchant, no matter how important carefully kept accounts may be to the success of a bank or mercantile business. It is not the surgeon who has the finest set of instruments, or even the greatest knowledge of their uses, who is necessarily the most success-

ful. It is not the lawyer who has read the most law, but the one who can discriminate and apply his knowledge to the case in hand, who is the best jurist. So it is not the engineer who is the most expert mathematician, the finest draftsman, or who may design and calculate the most beautiful structure, who is the most successful.

The true engineer of to-day is the one who, seeing a necessity arising in the onward march of civilization, can think out its solution, conceive a project, design the necessary works connected therewith, and carry out the scheme as a whole to a successful issue; at the same time convincing his fellow-men of the necessity for the work, the efficiency and economy of his design, and his own ability to accomplish the desired result.

The surplus labor of the past is the capital of to-day. The conservation and preservation of that capital is a necessary factor in promoting modern civilization.

Our works should increase the comforts of mankind, "cause two blades of grass to grow where one grew before," enable persons and property to be carried greater distances in shorter periods of time, with increased comfort and convenience and at the least possible expense. The economy of labor should be the ultimate object to be attained.

The best engineer is not necessarily the one who will design and construct an elaborate bridge across a mighty river, but the one who will design and construct such bridge so as to give the greatest amount of facility for transportation over it at the least possible expense. I have in mind a bridge over a great stream in America which, with its auxiliaries, was constructed some years ago at an expense approximating ten million dollars. It brought a national reputation to its engineers. I have in mind another bridge, located within five miles of the same structure, which, so far as railroad traffic is concerned, performs the same functions but cost one-tenth the amount of money. The construction of this latter bridge was a mere incident in the career of the engineers who designed and superintended the work, and it has brought practically no addition to their fame.

In comparing the works of engineers, that engineer is the best who designs and constructs, results being equal, in such a way that the interest on capital invested, together with maintenance charges and

operating expenses, are a minimum. It is certainly poor engineering to construct works so massive and with such a surplus of strength and solidity that the interest on the original amount invested far exceeds the cost of repairs, renewals and interest on a differently designed work which would perform a similar service.

In this I think the engineers of the Old World can learn from those of the New. On the other hand, in the construction of works of a permanent nature, so as to reduce the cost of operation, maintenance or renewal, the New World can certainly learn from the Old. During the past century, however, the problems which have confronted the engineers of the Old and New Worlds have been vastly different. Works have been constructed in the Old World to meet existing and stable conditions; in the New World, works have been provided to create conditions; consequently, it has been necessary for engineers in the New World to have not only vivid and correct imaginations, and to be able to judge of possibilities and future requirements, but to design and construct their works in such a way as to bring about the desired results with a limited amount of labor and capital.

It has not been a question of choice between what was best or even most economical; the main question has been one of what it was possible to do under the circumstances. Frequently, it was a case of the engineer creating the circumstances. Conditions, however, are rapidly changing, and the engineer of the New World is following closely in the footsteps of the Old World engineer, and his works are gradually becoming more substantial and permanent.

While it is not only desirable, but necessary, that the independent units of our profession, in the form of the individual engineer, should reach the highest state of efficiency, it is also necessary for the highest success that these units be combined. The closing century has witnessed gigantic combinations in the financial and industrial interests of the world. In the engineering profession, the engineers of the British Empire have organized the Institution of Civil Engineers and made it the leading engineering organization of the civilized world. The last half of the century has seen the birth and vigorous growth of our own organization, the American Society of Civil Engineers. Engineering organizations in France, Germany and other countries have also brought engineers together and tended to advance the standard of our profession.

In America, numerous local organizations have been perfected at secondary centers of population, which have done much to furnish engineers within their environment with the encouragement and strength due to association with their fellows. A strong tendency has also existed to form societies in the line of special engineering. These different organizations all have their functions. They are, however, restricted in their influence, and while engineers may be and should be benefited by a connection therewith, they should not forget that the American Society of Civil Engineers construes the term "Civil Engineer" in its broadest sense, and embraces all specialties and subdivisions of the engineering profession. While a connection with local or special organizations may benefit the individual engineer, the greater measure of benefit to the profession at large can be derived from affiliation with the more representative national body. It is through the national organization that we must expect to secure proper recognition for our profession.

Engineers in America do not appreciate the importance of this subject as fully as they should, or as much as their English brethren do, but they are becoming rapidly educated in this direction. As an instance of this, several years ago an engineer was sent to Australia in the interest of an American bridge company. He asked for a hearing before a certain commission which had charge of an important piece of engineering work. The first question asked him was, whether he was a member of the Institution of Civil Engineers of Great Britain. He replied that he was not. He was then asked if he was a member of the American Society of Civil Engineers. He gave a like answer, after which there fell upon the conference a silence that impressed him with the fact that he would have no standing as an engineer in that country unless he was connected with one or the other of these great bodies. He immediately cabled his principals that it would be impossible for any representative of their company to succeed in that quarter of the globe as an engineer unless he was affiliated with one of the national organizations.

I cite this instance in order to impress not only upon the members of the Society but upon American engineers generally, the importance of the fact that these organizations have a higher standing in the eyes of the lay public than they sometimes have in the eyes of the individual engineer.

The point I desire to make and emphasize is the importance and responsibility of the engineering profession in relation to the advancement of civilization, and the fact that it is our duty, first, to fit ourselves as individuals in every possible way for the responsibility placed upon us, and secondly, that we should combine individual units in organization; that it is the duty of the engineer first to fit himself for his individual work, and next to combine with his fellow engineers for the strengthening and elevating of the profession.

From time to time much has been said in relation to engineering ethics, and suggestions have even been made that they be formulated. We should not forget that engineering ethics are simply business ethics, and that the foundation of all ethics is the golden rule, which we should individually endeavor to observe and practice.

This year and to-day will mark an epoch in the history of the American Society of Civil Engineers. This Society holds the same relation to America that the Institution of Civil Engineers holds to the British Empire. With grateful appreciation and profound respect we have accepted the invitation of the Institution of Civil Engineers, and hold our Thirty-second Annual Convention in this house, the home of the greatest engineering association in the world; and we feel warmly the kindness and sympathy which this invitation implies upon the part of our elder brothers. We hope that this meeting will have the effect of not only strengthening the bonds between the different branches of the Anglo-Saxon race, to which most of our members belong, but also of strengthening the larger bond which should unite all mankind and bring about the ultimate acknowledgment of the common brotherhood of man. This grand consummation of our labors, whether we realize it or not, is the ultimate destiny of our profession.

MEMOIRS OF DECEASED MEMBERS.

POMEROY P. DICKINSON, M. Am. Soc. C. E.*

DIED OCTOBER 4TH, 1895.

Pomeroy P. Dickinson was born in Rochester, N. Y., March 15th, 1827, his father, Patrick Peebles Dickinson—a descendant of the Colonial Dickinsons, of Massachusetts—having removed to that city from Amherst, Mass., in the year 1814.

His education was derived from the schools of the period, and finished at the Rochester University, under Dr. Dewey, in 1843. His tastes led him to adopt the profession of engineering, his first experience having been gained in 1845, on the Newburg Branch of the New York and Erie Railway. A diligent student, a persistent and conscientious worker, he rapidly rose through the various grades, reaching at an early age a position which marked him as a leader in his chosen profession. He held many positions of honor and trust, among them being that of Chief Engineer of the following enterprises: Eagleton Coal and Railroad Company, of Pennsylvania; Poughkeepsie and Eastern Railroad; Columbia County Iron Company; Poughkeepsie Bridge Company; Northern Central Railroad (from Baltimore to Sunbury, Pa.); New York and Richmond Coal and Railroad Company, of Virginia, and New York and Long Island Railroad Company. He was also interested in the construction of the main line between Port Jervis and Dunkirk; the distributing reservoir for Poughkeepsie City; the Pontiac Branch Railroad, in Rhode Island; the Atlantic and Richmond Air Line Railroad; the Savannah and Memphis Railroad, in Alabama; the Elberton Air Line Railroad of Georgia; the Cincinnati and S. E. Railroad, of Kentucky, and many other prominent roads.

In 1887–1890, he prepared plans and estimates of the cost of the New York Arcade Railroad, which were examined by engineers of French and German capitalists, sent to this country for this special purpose; the plans and estimates were fully approved by this commission, and the money needed for the first 6 miles of the Arcade road (under Broadway from the Battery to Fifty-ninth Street, with a branch from Twenty-third Street under Madison Avenue to Forty-second Street Depot) was contracted to be furnished; two-thirds of the cost, which was estimated at $20 000 000, was to be furnished by French and

* Memoir prepared by Edward A. Greene, M. Am. Soc. C. E.

German capital, and one-third by American capital. These contracts depended upon the decision of the Court of Appeals of New York State, as to the constitutionality of the charter. The declaring of the charter by the Court as unconstitutional, defeated the early beginning of what would have resulted, certainly before this time, in completed underground rapid transit from the Battery to the Harlem River.

Mr. Dickinson's prominence was not confined to business or professional circles; his social and benevolent relations were extended and widely diffused. A devout member of the Episcopal Church, he led a sincere Christian life. He was senior warden of St. Ann's Protestant Episcopal Church, delegate of the Diocese of New York to the Episcopal Convention, trustee and treasurer of the Endowment Society of St. Ann's Church, vice-president and trustee of the Church Mission to Deaf Mutes, Chairman of the Executive Committee of the Board of Trustees of the Gallaudet Home for Infirm and Aged Deaf Mutes, and one of the Board of Managers of the Protestant Episcopal City Mission Society. He was also a member of the Colonial Club.

As a man he was beloved by all who knew him—of a disposition genial and hearty, yet withal modest and retiring. In the young men of his profession he manifested the deepest interest, extending at all times kindly encouragement, and, when necessary, a helping hand. He recognized to the fullest degree, the brotherhood of mankind, the breadth of his sympathies being attested by his regular Sunday visits to the "Tombs," where he read and talked to the inmates, putting hope into the heart of many an unfortunate, who to-day blesses his memory.

His life was based upon high ideals, up to which he conscientiously lived. His mind was cast in an analytic mould, and whether in the solution of professional problems or the pursuit of abstract knowledge, the love of truth was the incentive to research. He was one of "Nature's Noblemen," and the world is better for his having lived.

In 1853, Mr. Dickinson married Miss Charlotte H. Kirby, who survives him.

Mr. Dickinson was elected a Member of the American Society of Civil Engineers, January 17th, 1872.

ROBERT GILLHAM, M. Am. Soc. C. E.*

DIED MAY 19TH, 1899.

Robert Gillham was born in New York City, September 25th, 1854, being the third son of John and Clarissa Gillham. His father is an Englishman, but emigrated to America in early life, and has held important positions of trust under the Government of the United States.

His mother was an American, from one of the oldest and most respected families of New Jersey.

Having completed a course at a private school in Lodi, N. J., at the age of sixteen, he entered the Institute at Hackensack, N. J. He soon became assistant to Professor Williams, President of the Institute, and under his private instructions continued the study of engineering until 1874, when he began to practice his profession at Hackensack. Here, many problems were entrusted to him, the successful solution of which brought him much special work from New York City.

It was at this time that he was given the problem of utilizing the sulphur in zinc ores. After a most exhaustive investigation of all the methods in use, Mr. Gillham designed a furnace for desulphurizing zinc ores in such a way that, while the value of the zinc was in no way affected, the sulphuric acid gas was used in the manufacture of sulphuric acid. This was a most important step in the economy of zinc production.

Mr. Gillham removed to Kansas City, Mo., in 1878. He quickly saw the advantages of cable traction for the operation of street railways for that city, and at once proceeded to make plans for a cable railway to run from the Union Depot, up the precipitous bluff, direct to the business center of the city. The idea of such a thing was so novel, and the engineering difficulties were apparently so great, that his project was first regarded as chimerical by all those who were in a position to assist him. After many disappointments and discouragements, he succeeded finally in enlisting the necessary capital. His next problem was to obtain a franchise from the city. Here, he encountered the resistance of the horse-railway company, which, notwithstanding its inadequacy, was being operated at a great profit. This resistance, owing to the great local influence of that company, proved, according to Mr. Gillham's own statement, one of the most formidable obstacles encountered in his career. However, with an

* Memoir prepared by R. J. McCarty, M. P. Paret and G. W. McNulty, Members, Am. Soc. C. E.

unexampled tenacity of purpose, he persisted until success had crowned his efforts.

The spectacle of this young man of twenty-five, practically a stranger in a strange land, essaying to solve engineering problems which older heads had pronounced impracticable, proceeding with his task, undisturbed by the strictures of ignorance and undismayed by the immense power of municipal politics which he found arrayed against him, and conducting his enterprise to a pre-eminently successful issue, both in a physical and a financial sense, may justly be held up to all men, both young and old, for emulation.

The complete and astonishing success which finally attended his labors was emphasized, not only by the enrichment of his associates, but also by the adoption of his plans, to its immense profit, by the very company which had so bitterly opposed him.

It is sad to relate, however, that just in the hour of his triumph he was stricken down by an accident which caused him to hover between life and death for many months, when he should have been resting in peace from his labors and enjoying in health and comfort the feeling which comes from the consciousness of important services rendered and arduous duties well performed.

Having recovered his wonted health and spirits, he conceived the idea of an elevated railway over the low lands to the west of Kansas City, Mo., and across the Kansas River to Kansas City, Kans.

Owing to the then undeveloped condition of street-railway traction, he was forced to the use of steam, it not being practicable to use the cable. Having constructed the road west from the Union Depot in Kansas City, Mo., it soon became necessary to extend it eastward to the business center of that city. This called for a tunnel of 700 ft. under the bluff, on a 9% gradient, and made necessary the adoption of cable traction for the extension. The whole work was completed in 1887, but the road, from the necessity of having two kinds of power, and from the fact of its being a little in advance of requirements, did not prove at first a financial success.

About this time he also had charge of the construction of the Omaha Cable Railway, the Denver City Cable Railway, the Montague Street Cable Railway, in Brooklyn, and the Cleveland City Cable Railway, all of which enterprises were based upon the success of Mr. Gillham's Kansas City Cable Railway.

In 1888 he was retained to investigate the problem of changing the motive power of the Boston street railways from horse to cable.

Before he had completed his investigations, however, the practicability of electricity as a motive power for street railways had been demonstrated, and the Boston people decided to adopt that method of traction.

Shortly after this, in connection with the late John A. Wilson, M.

Am. Soc. C. E., of Philadelphia, he made a report relating to an extensive elevated railway system for Boston.

He also visited Europe, and made extensive investigations of the problem of compressed air, in Paris and London.

In 1890 the Kansas City Elevated Railway Company had become involved in financial difficulties, and Mr. Gillham was asked to take charge of the rehabilitation of the property, and was appointed Receiver and General Manager of that company. He at once changed the motive power to electricity, and by his successful management and adroit and able conduct was able, in 1894, to effect a sale of the properties to the Metropolitan Street Railway Company, on terms highly advantageous to those he represented.

He was at the same time Receiver and General Manager of the North East Street Railway Company, of Kansas City, and conducted the affairs of that company to a highly satisfactory settlement.

Mr. Gillham's labors in connection with the receivership and management of the Kansas City Elevated Railway were exceedingly arduous, so much so, in fact, that in the summer of 1894, shortly after the settlement of that company's affairs, he was stricken with nervous prostration, and was compelled to spend some months in recuperation.

In 1895, he accepted the position of Chief Engineer of the Kansas City, Pittsburg and Gulf Railroad Company. At that time this railroad had only been built as far as Siloam Springs, Ark., 230 miles south of Kansas City. By the latter part of 1897, Mr. Gillham had the line completed and in operation through to Port Arthur, Tex., a total distance of nearly 800 miles from Kansas City, and had also built and in operation, about 90 miles of railroad in Missouri, north from Kansas City, and 50 or 60 miles of branch roads in the southern parts of the Pittsburg and Gulf Railroad System.

One great work which Mr. Gillham undertook, was the construction of the Port Arthur Ship Canal, which extended from Port Arthur, Tex., the southern terminus of the Pittsburg and Gulf Railroad, to deep water at Sabine Pass, Tex., on the Gulf of Mexico. This canal is 7½ miles long, 175 ft. wide and 25 ft. deep, and at the time of writing this memoir, but a few months after Mr. Gillham's death, is practically completed. The canal was begun in the latter part of 1896. The engineering difficulties were readily overcome by the foresight and resource of its Chief Engineer, though the opinions of several well-known engineers were against the practicability of the project.

From the first, the enterprise met with strenuous opposition from adverse interests, and the opponents of the canal thoroughly exhausted every known method of resistance, both in and out of the courts. They even succeeded in getting the United States Government to interfere. Mr. Gillham was, however, not only an engineer, but a diplomatist of more than average ability, and having, moreover, a remarkable faculty

of presenting his views and opinions in a plain and convincing manner, he soon broke down all opposition.

In 1896, he was appointed General Manager of the Kansas City, Pittsburg and Gulf Railroad, as well as of the Kansas City Suburban Belt Railroad.

In 1897, in addition to these positions, he also assumed the positions of General Manager and Chief Engineer of the Omaha and St. Louis and the Omaha, Kansas City and Eastern Railroad, and of the Kansas City and Northern Connecting Railroads.

On April 1st, 1899, he was appointed one of the Receivers of the Kansas City, Pittsburg and Gulf Railroad, retaining his position as General Manager both of the Kansas City, Pittsburg and Gulf and the Kansas City Suburban Belt Railroads, but resigning his position on the other roads mentioned.

On April 27th, 1899, as a result of the litigation of the affairs of the Kansas City, Pittsburg and Gulf Railroad, a change was made in the receivership of that company, but Mr. Gillham was, in recognition of his services and abilities, appointed by the Court as General Manager for the new receivers.

But this sphere of routine railroad work, though large, not being sufficient to satisfy his enterprising spirit, he at the same time sought other fields to fill the measure of his energy and talents.

Thus, at the time of his death, he held the following positions:

General Manager and Chief Engineer for the Receivers of the Kansas City, Pittsburg and Gulf Railroad; General Manager and Chief Engineer of the Kansas City Suburban Belt Railroad; General Manager and Chief Engineer of the Port Arthur Channel and Dock Company; President of the Armourdale Foundry Company; Vice-President of the Kansas City Elevated Railway Company; Director in the Missouri, Kansas and Texas Trust Company and the Kansas City, Pittsburg and Gulf Railroad.

He was also an active member of the Board of Park Commissioners of Kansas City, Mo., in which capacity he had done much to develop the park systems of that city.

He was a leading member of the Commercial Club of Kansas City, having, as such, devoted much of his time and energies to the work of that organization.

He was a member of the American Society of Civil Engineers, of the Society of Marine Architects and Naval Engineers, and of the Institution of Civil Engineers of England.

In December, 1881, he was married to Miss Minnie Marty, the daughter of a prominent capitalist of Kansas City. His wife, two daughters and an infant son survive him.

For some time before his death, the condition of his health admonished his friends that he was overworked, in fact, during this

period it would seem that nothing save his enterprising spirit and wonderful recuperative powers had borne him up.

On May 13th, 1899, having just returned from an exhausting tour of inspection over the Kansas City, Pittsburg and Gulf Railroad, he left his office at 6 P. M. as usual. As soon as he reached home he was stricken with a nervous chill, which was followed by pneumonia. His physicians stated that ordinarily he would have easily thrown off the disease, but that in his exhausted condition the worst might be feared. He lingered until 8.30 P. M., May 19th, when he passed peacefully away, surrounded by his family and nearest friends.

Mr. Gillham's interesting and valuable career resulted from a rare faculty to create his own opportunities, from an ability to use the same for the achievement of practical results, from an integrity and loftiness of purpose which directed his powers to the highest and most useful ends, from a genial and engaging manner, concerning the sincerity of which his contempt for all hypocrisy and his many deeds of disinterested kindness left no room for doubt, and from a courage, constancy and energy which left him ever undismayed.

It happened that just before his death he was selected to present a tribute to the memory of a prominent fellow citizen. This tribute contained the following words:

"Nature, having so constructed man that he might not exist without relation to his fellow man, has kindly placed among us some whose mission is to scatter peace and happiness all along the path of life. To every one of these there is a monument built of his own good deeds, and though inscriptions may not be upon it, God will know to whom it was erected."

These words will serve to show the standard of their author's excellence, and his life and deeds proclaim how well he followed it.

Mr. Gillham was elected a Member of the American Society of Civil Engineers, June 2d, 1886.

HORACE HARDING, M. Am. Soc. C. E.*

DIED JULY 29TH, 1899.

Horace Harding was born at Boston, Mass., on May 27th, 1828. He was graduated from Harvard University in July, 1848, and at once began the practice of his chosen profession. His first work was an elaborate survey of Springfield Cemetery, after completing which he joined a party of engineers under Captain John Childe, of Springfield, and went South to take part in the surveys for the Mobile and Ohio Railroad in Alabama and Mississippi. He remained with this company until 1854, and then became Chief Engineer of the Indianapolis and Toledo Railroad, in Ohio. He returned to the engineer corps of the Mobile and Ohio Railroad in 1856, and in the following year went to Missouri, where he combined the real estate business with professional work. He returned to Alabama in 1858, and joined the engineer corps of the North-East and South-West Alabama Railroad.

Mr. Harding married, on May 31st, 1859, Miss Eliza P. Gould, daughter of William P. Gould, a prominent planter of Greene County, Ala. He remained with the North-East and South-West Alabama Railroad until it suspended work, in the latter part of 1860, when he went to Mobile and built one or more street railroads in that city.

On the outbreak of the civil war Mr. Harding enlisted in the 20th Alabama Regiment. In April, 1862, he was detailed by the Confederate Government as Superintendent of Road Repairs on the Mobile and Ohio Railroad. He remained in that capacity until 1871, and became General Superintendent of the Alabama and Chattanooga (now A. G. S.) Railroad in 1872.

In June, 1874, Mr. Harding entered the Government service as United States Assistant Engineer on the Improvement of the Warrior River, Ala., and remained in charge of this important work for twenty-two years, until May, 1896, when he resigned his position and retired from active life, thus closing a long and busy professional career, about which the least that can be said is "well done, thou good and faithful servant." During the last years of his engineering work he designed and built in the Warrior River, at Tuscaloosa, Ala., at a cost of over half a million dollars, three handsome masonry locks and dams, which will stand as an enduring monument to his professional ability.

He spent the closing years of his life in Birmingham, Ala., and died of typhoid fever after a short illness, while on a visit to relatives

* Memoir prepared by R. C. McCalla, M. Am. Soc. C. E.

in Grand Rapids, Mich. He leaves two sons, William P. G. Harding and Captain Chester Harding, Corps of Engineers, U. S. A.

Mr. Harding was a son of Chester Harding, a noted artist of Boston, who painted many famous portraits both here and abroad during the first half of this century.

Mr. Harding was a man of rare qualities of mind and heart, and enjoyed the confidence and esteem of all who knew him. An able engineer, a talented inventor, a versatile writer; upright, generous and gentle to a fault; he lived and died without a selfish thought, and with a charity as broad as the Universe. Were there more like him the world would be better off.

Mr. Harding was elected a Member of the American Society of Civil Engineers, November 2d, 1892.